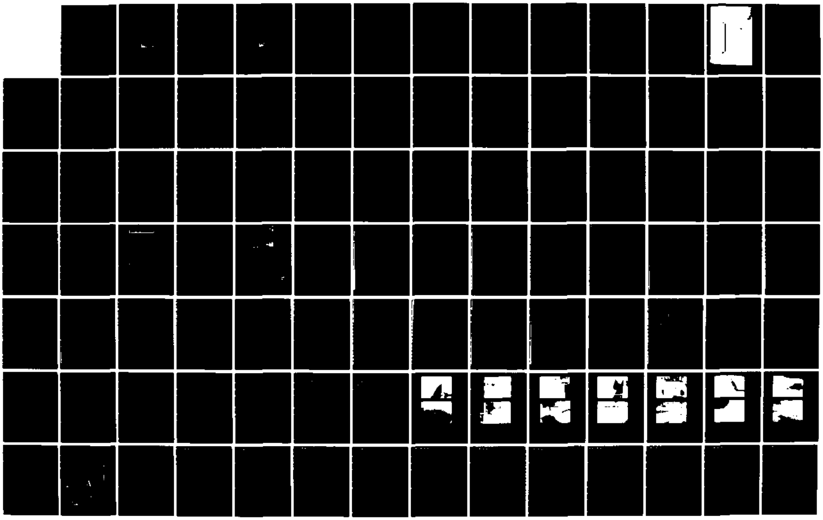
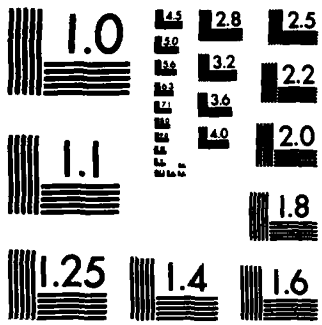
AD-A144 665 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS COLLINS COMPANY LOWER..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV JUL 79 1/2

UNCLASSIFIED F/G 13/13 NL




1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0

CONNECTICUT RIVER BASIN

AVON-BURLINGTON, CONNECTICUT

COLLINS COMPANY LOWER DAM
CT 00380

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

JULY, 1979

84 08 20 059

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER CT 00380	2. GOVT ACCESSION NO. AD-A144665	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Collins Company Lower Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE July 1979
		13. NUMBER OF PAGES 80
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut River Basin

Avon-Burlington, Connecticut

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This facility consists of a 400 foot long concrete gravity dam across the Farmington River consisting of a 40 foot long right abutment housing three low level sluice gates, a 300 foot long spillway, and a 60 foot long left abutment with a brick gatehouse housing six intermediate level sluice gates. The hydraulic height of the dam is approximately 33 feet. Based upon the visual inspection at the site, existing data, and past performance, the dam is judged to be in fair condition. The small size and significant hazard classification of this dam, the test flood will be equivalent to ½ the PMF.

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

CONNECTICUT RIVER BASIN

AVON-BURLINGTON, CONNECTICUT

COLLINS COMPANY LOWER DAM
CT 00380

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

JULY, 1979

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	COLLINS COMPANY LOWER DAM
Inventory Number:	CT - 00380
State Located:	CONNECTICUT
County Located:	HARTFORD
Towns Located:	AVON AND BURLINGTON
Stream:	FARMINGTON RIVER
Owner:	STATE OF CONNECTICUT
Date of Inspection:	APRIL 26, 1979
Inspection Team:	PETER M. HEYNEN, P.E.
	THEODORE STEVENS
	GONZALO CASTRO, P.E.
	CHARLES OSGOOD

This facility consists of a 400 foot long concrete gravity dam across the Farmington River consisting of a 40 foot long right abutment housing three low level sluice gates, a 300 foot long spillway, and a 60 foot long left abutment with a brick gatehouse housing six intermediate level sluice gates. The hydraulic height of the dam is approximately 33 feet. The six sluices in the left abutment discharge to a 640 foot long canal feeding a powerhouse at the end of the canal, which is not presently in use. The canal was excavated in the left riverbank and is lined with a concrete wall on the right side and a masonry wall on the left side.

Based upon the visual inspection at the site, existing data, and past performance, the dam is judged to be in fair condition. No evidence of immediate structural instability of the dam or its appurtenances was observed, however, there is spalling of concrete on the abutments, canal walls, powerhouse bulkhead, and most probably on the crest and downstream face of the spillway and there is a seep at the juncture of the right abutment of the dam with the adjacent bedrock riverbank.

In accordance with Corps of Engineers guidelines for the small size and significant hazard classification of this dam, the test flood will be equivalent to one-half the Probable Maximum Flood (PMF). Peak inflow to the dam impoundment is 83,000 cubic feet per second (cfs); peak outflow is 83,000 cfs with the dam overtopped 8 feet. The spillway capacity to the top of the dam is 33,000 cfs, which is equivalent to approximately 40% of the routed test flood outflow.

It is recommended that the owner retain the services of a registered professional engineer to inspect the downstream face of the dam under non-overflowing conditions with the upstream water level near the spillway crest elevation, and make recommendations for the repair of any problem conditions discovered. Based upon his findings, the engineer should then determine if a structural analysis of the dam based upon field measurements of actual uplift pressures and determinations of actual foundation conditions, is necessary.

The above recommendations and any remedial measures discussed in Section 7, should be instituted within one year of the owner's receipt of this report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

STATE OF CONNECTICUT
PETER M. HEYNEN
NO. 9618
REGISTERED
PROFESSIONAL ENGINEER

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.

STATE OF CONNECTICUT
EDGAR B. VINAL, JR.
No. 4158
REGISTERED
PROFESSIONAL ENGINEER

This Phase I Inspection Report on Collins Company Lower Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

SAUL C. COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

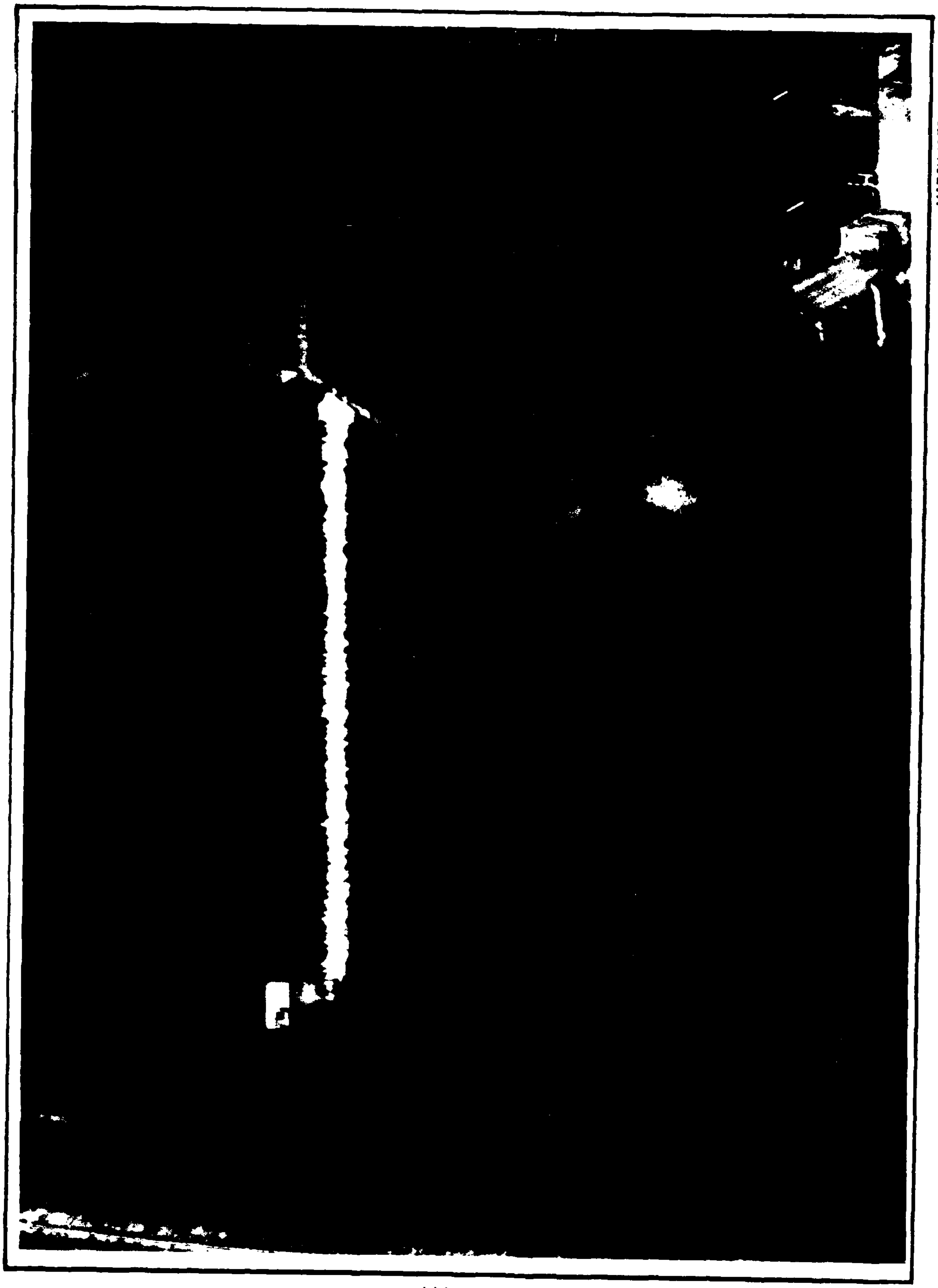
In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

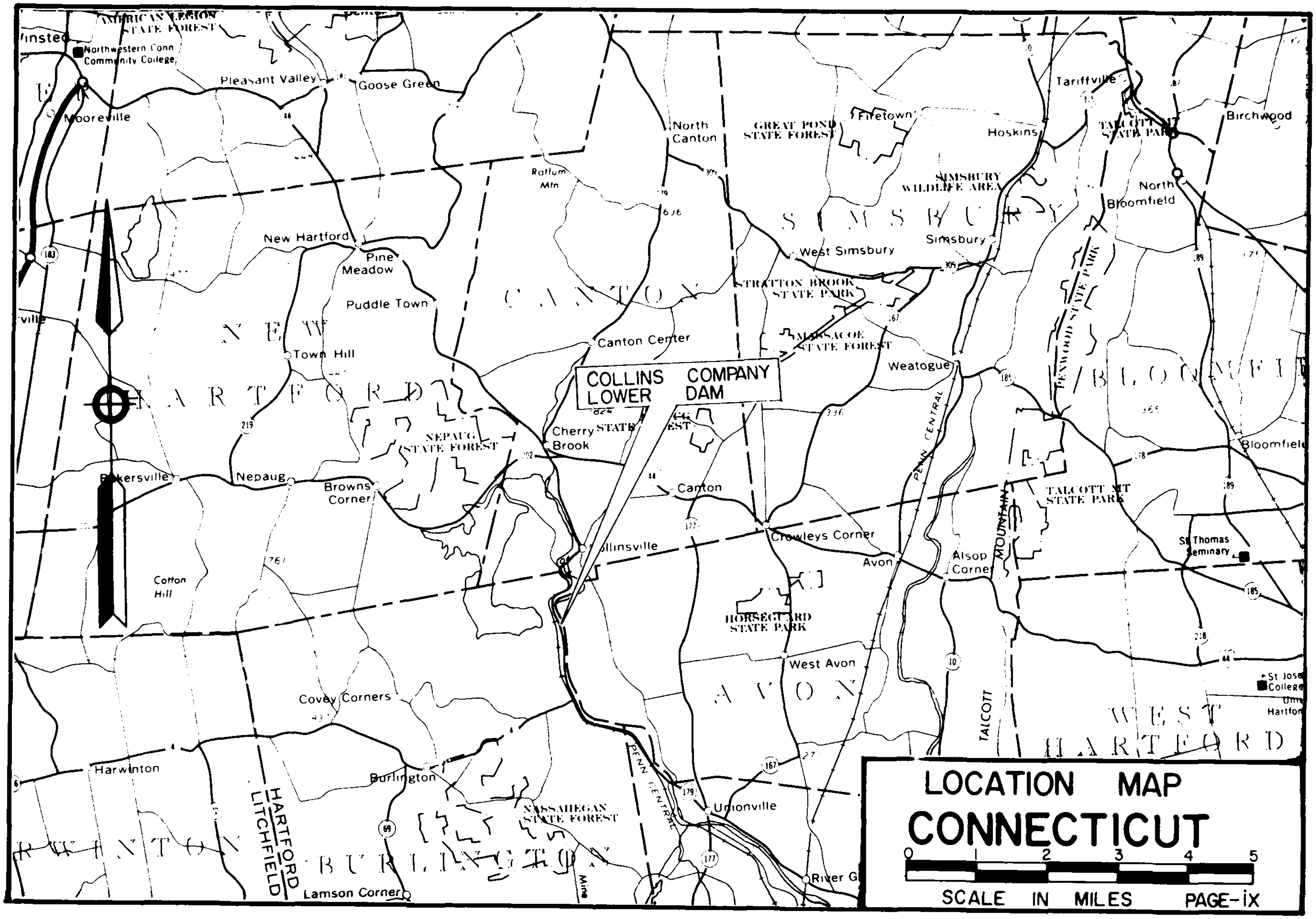
Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

TABLE OF CONTENTS

APPENDICES



MARCH, 1979


COLLINS COMPANY LOWER DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5
SCALE IN MILES
New Hartford
Pine Meadow
Puddle Town
Town Hill
Pleasant Valley
Goose Green
Mooreville
Bakersville
Nepaug
Browns Corner
Cotton Hill
Covey Corners
Harwinton
Burlington
Lamson Corner
HARTFORD
LITCHFIELD
NEPAUG STATE FOREST
Ratlum Mtn
Canton Center
Cherry Brook
Canton
Collinsville
North Canton
West Simsbury
Simsbury
Weatogue
Tariffville
Hoskins
Firetown
Birchwood
North Bloomfield
Bloomfield
GREAT POND STATE FOREST
SIMSBURY WILDLIFE AREA
STRATTON BROOK STATE PARK
MASSACOE STATE FOREST
TALCOTT MT STATE PARK
PENWOOD STATE PARK
Crowleys Corner
Avon
Alsop Corner
West Avon
Unionville
River Glen
HORSEGUARD STATE PARK
NASSAHEGAN STATE FOREST
PENN CENTRAL
TALCOTT MOUNTAIN
St Thomas Seminary
St Joseph College
Univ Hartford
Northwestern Conn Community College
AMERICAN LEGION STATE FOREST
CANTON
SIMSBURY
AVON
WEST HARTFORD
NEW HARTFORD
BURLINGTON
Mine

PAGE-IX

PHASE I INSPECTION REPORT

COLLINS COMPANY LOWER DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of November 28, 1978 from Max B. Scheider, Colonel, Corps of Engineers, Contract No. DACW 33-79-C-0014 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on the Farmington River in a rural area of the towns of Avon and Burlington, County of Hartford, State of Connecticut. The dam is shown on the Collinsville USGS Quadrangle Map having coordinates latitude N 41° 48.0' and longitude W 72° 55.7'.

b. Description of Dam and Appurtenances - The concrete gravity structure across the Farmington River, reportedly keyed to bedrock a maximum of 51 feet below the top of the dam, is 400 feet long with an adjacent canal and hydroelectric power generation facility. The dam, with a structural height of 51 feet, rises about 33 feet above the downstream riverbed. It consists of a 40 foot long right abutment with a top elevation of 276.7, a 300 foot long spillway with a rounded crest at elevation 264.7 and a 60 foot long left abutment at elevation 276.7. Three 4' x 4' low level sluice outlets at invert elevation 249.2 are located in the right abutment with manually cranked floor stands in a wood beam and corrugated sheet metal gatehouse atop the abutment. Intake to the approximately 640 foot long powerhouse canal at the left end of the dam is through six 6' X 8' sluice openings at invert elevation 254.7. These sluice gates are controlled by manually operated worm gear hoists in a brick gatehouse on the left abutment. There is an approximately 400 foot long concrete retaining wall along the right side of the impoundment upstream of the dam and an 115 foot long curved retaining wall at the left end upstream of the dam, creating a small bay upstream of the gatehouse.

The canal, excavated from the previously existing riverbank, is bounded on the right by a concrete retaining wall with a top elevation of 271.2 and on the left by a dry laid stone retaining wall. A non-operational powerhouse at the downstream end of the canal consists of a brick superstructure atop a concrete substructure. There is a 50 foot waste weir at elevation 268.7 and low level canal discharge sluice gate immediately upstream of the powerhouse and adjacent to the concrete canal wall. Intake to the powerhouse is through wood slide gates and discharge is into a tailrace leading to the river channel immediately downstream of the powerhouse where there is a granite-gneiss masonry retaining wall along the riverbank.

c. Size Classification - SMALL - The dam impounds a maximum of 690 acre-feet of water with the river level at the top of dam elevation, which is approximately 33 feet above the elevation of the riverbed downstream of the dam. According to the Recommended Guidelines, this dam is classified as small in size.

d. Hazard Classification - SIGNIFICANT - Although the area downstream of the dam is undeveloped, there is heavy recreational usage of the Farmington River in this area and a breach of the dam with the water level at the spillway crest yields potential for loss of life for several miles along the river downstream of the dam.

e. Ownership - The dam was originally built and owned by the Collins Company, which, in 1966, sold the facility to the Hartford Electric Light Company. H.E.L.Co. shortly thereafter turned ownership over to the State of Connecticut Department of Environmental Protection.

State of Connecticut
Department of Environmental Protection
Region 1 Headquarters
P. O. Box 161
Pleasant Valley, CT 06063
Mr. Anthony Cantelle (203) 379-0771

f. Operator - None

g. Purpose of Dam - The dam was originally constructed as a hydroelectric power generation facility, but is no longer used for this purpose. The dam impoundment is used for recreational purposes.

h. Design and Construction History - The following information is believed to be accurate according to the available plans. The dam and appurtenances were constructed during 1912 and 1913 by the Collins Company as designed by Edwin P. Ball, Engineer. The small wood beam and corrugated metal gatehouse at the right end of the dam is not shown on the original drawings and was probably added at a later date. One foot of concrete was added to the canal waste weir in 1965 as shown on drawings dated September, 1965. The drawings also show a one foot addition of concrete previous to the 1965 resurfacing. There is no available documentation or evidence of other construction work done at the site.

i. Normal Operational Procedures - There are no operational procedures followed for the dam.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area is 360 square miles of largely undeveloped rolling to mountainous terrain of which 236 square miles are (partially) controlled by three flood control projects as well as four other dams. The flood control projects, Colebrook River Dam, Mad River Dam and Sucker Brook Dam, regulate approximately 140 square miles of the watershed. The other dams, which regulate approximately 96 square miles, are Highland Lake Dam, Saville Dam, Richard's Corner Dam and Nepaug Reservoir Dam. The Collins Company Upper Dam, directly upstream from the Collins Company Lower Dam, has no peak inflow reducing effect on the Lower Dam.

b. Discharge at Damsite - Discharge is over the 300 foot long spillway, through three 4' x 4' low level gates in the right abutment, through six 6' x 8' sluice gates in the left abutment to the canal, over the 50 foot long canal waste weir, through the 3' x 3' canal low level sluice, and through the two 9' x 14' intake sluices of the powerhouse.

1. Outlet Works (Conduits):	3 low level outlets @ invert el. 249.2
	6 canal intake sluices @ invert el. 254.7
	1 canal low level sluice @ invert el. 254.7
	2 powerhouse intake gates @ invert el. 255.7
2. Maximum known flood at damsite:	105,000 cfs, Aug., 1955
3. Ungated spillway capacity at top of dam el. 276.7:	33,000 cfs.
4. Ungated spillway capacity at test flood el.:	N/A
5. Gated spillway capacity at normal pool el.:	N/A
6. Gated spillway capacity at test flood el.:	N/A
7. Total spillway capacity at test flood el.:	N/A
8. Total project discharge at test flood el. 285:	83,000

c. Elevations (Feet Above Mean Sea Level)

1. Streambed at center-line of dam:	244±
2. Maximum tailwater:	285±, Aug., 1955
3. Upstream portal invert diversion tunnel:	N/A
4. Recreation pool:	264.7
5. Full flood control pool:	N/A

6. Spillway crest: 264.7

7. Design surcharge (original design): N/A

8. Top of dam: 276.7

9. Test flood design surcharge: 285±

d. Reservoir

1. Length of maximum pool: N/A

2. Length of recreation pool: N/A

3. Length of flood control pool: N/A

e. Storage

1. Recreation pool: 160 acre-ft.

2. Flood control pool: N/A

3. Spillway crest pool: 160 acre-ft.

4. Top of dam: 690 acre-ft.

5. Test flood pool: 690 + acre-ft.

f. Reservoir Surface — N/A - Run-of-river dam. (See Appendix D-2, Storage)

g. Dam

1. Type: Concrete gravity

2. Length: 400 ft.

3. Height: 51 ft. structural
33 ft. hydraulic

4. Top width: 24 ft. left abutment
8 ft. right abutment

5. Side slopes: N/A

6. Zoning: N/A

7. Impervious core: N/A

8. Cutoff: N/A

9. Grout curtain: N/A

10. Other: N/A

h. Diversion and Regulating Tunnel - N/A

i. Spillway

1. Type:	Concrete broad-crested weir of trapezoidal cross-section approximating an ogee section.
2. Length of weir:	300 ft.
3. Crest elevation:	264.7
4. Gates:	None
5. Upstream Channel:	Riverbed
6. Downstream Channel:	Riverbed
7. General:	Able to accomodate 5' flashboards

j. Regulating Outlets

Three low level outlets

1. Invert:	249.2
2. Size:	4' x 4'
3. Description:	Sluice gates
4. Control Mechanism:	Manually operated floor stands

Six canal intake gates

1. Invert:	254.7
2. Size:	8' x 6'
3. Description:	Sluice gates
4. Control Mechanism:	Manually operated worm gear hoist

Two powerhouse intake gates

1. Invert:	255.7
2. Size:	9' x 14'
3. Description:	Wooden slide gates
4. Control Mechanism:	Manually operated floor stands

One canal low level sluice

1.	Invert:	254.7
2.	Size:	3' x 3'
3.	Description:	Sluice gate
4.	Control Mechanism:	Manually operated floor stand.
5.	Other:	N/A

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Available Data - The available data consists of drawings for the original dam construction by Edwin P. Ball, Engineer, inventory data by the State of Connecticut D.E.P., and a "Reconnaissance Engineering Geologic Investigation" by Robert L. Nelson of Foundation Sciences Inc., which was incorporated into the "Canton Hydroelectric Project Feasibility Study" (CHPFS) by the Development and Resources Corporation (DRC).

b. Design Features - The drawings and reports indicate the design features noted in Section 1.

c. Design Data - A cross-section of the dam is included in the drawings by Edwin P. Ball showing resultant forces assuming the water level at spillway crest and with nine feet of water over the spillway. A stability analysis was performed in 1978 and included in the CHPFS assuming various loading and seismic conditions. The study, a portion of which is included in Appendix B, states:

> " A possible problem with regard to stability could exist since calculations indicate that the dams' overturning factors of safety are below normally expected values. In view of these low factors, it is apparent that some type of anchorage at the toe of these structures most probably exists... It is recommended that the magnitude of pressures at the toe and heel of each structure be checked by field testing to determine the magnitude of actual uplift forces. Further review and structural analysis of each structure should then be carried out based upon the observed uplift pressures and actual anchorage conditions."

2.2 CONSTRUCTION

a. Available Data - No as-built drawings of the dam or construction inspection records were available. Photographs taken during construction of the dam depicting construction methods and sequences, are in the possession of Mr. Thomas Perry of the T. M. Perry Company.

b. Construction Considerations - No information was available.

2.3 OPERATIONS

Flow in the Farmington River at gaging station Number 01187980, located approximately 750 feet downstream of the dam, has been recorded daily by the U.S.G.S. since November 1962. The Collins Company kept formal operations records during the years the dam was used for power generation, however in recent years (since 1966) no formal operations records are known to exist. On one occasion, the Water Resources Unit of the Connecticut Department of Environmental Protection did perform an inspection of the dam.

2.4 EVALUATION

a. Availability - Inventory data and an inspection report were provided by the Water Resources Unit of the Connecticut Department of Environmental Protection. Design drawings of the dam were provided by Mr. Thomas Perry of the T. M. Perry Company of Canton, Connecticut. The Reconnaissance Engineering Geologic Investigation and the CHPFS were provided by Mr. Dean C. Porterfield of the Canton Conservation Commission. The construction photographs were viewed by arrangement with Mr. Perry.

b. Adequacy - The nature of the engineering data available was sufficient to allow the Development and Resources Corporation to perform approximate stability analyses, however an assessment of the dam could not be based on this data. Therefore the final assessment of this dam is based on performance history, visual inspection, hydraulic computations of spillway capacity and approximate hydrologic judgements.

c. Validity - A comparison of the record data and visual observations reveals no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The general condition of the dam is fair. Inspection did reveal several areas requiring attention. The reservoir level was approximately six inches above the spillway crest at the time of our inspection. There were also several persons fishing immediately below the dam at the time of our inspection.

b. Dam

Spillway -The 300 foot long spillway is a concrete section keyed to rock with a rounded crest similar to that of an ogee section with pipe sockets along the crest, which would allow for installation of flashboards. The upstream face was underwater and could not be observed. The substantial flow over the spillway obscured observation of the crest and downstream face, however the turbulence of the flow over the spillway is an indication that the surface may be irregular due to spalling and cracking (Photo 1). Upon a subsequent inspection at the site when there was a slightly lower flow, aggregate was visible in the concrete at the crest, but the degree of deterioration of the concrete could not be determined (Photo 2). Also, extensive horizontal cracking along the downstream face was suggested by long lateral lines of white water which appear to be either splashing off of horizontal surfaces, water coming through the dam, or a combination of the two. The spillway apron could not be observed

Right Abutment - The 40 foot long right abutment, rising 12 feet above the spillway crest to elevation 276.7 houses three low level outlets. The abutment, also keyed to rock, has a vertical upstream face, an eight foot wide crest and a downstream face on a batter of two vertical to one horizontal. The downstream face of the abutment is heavily deteriorated, exhibiting significant spalling with clumps of grass growing from the concrete (Photo 4). The left end and upstream face of the abutment are in slightly better condition than the downstream face, but are, nonetheless, significantly spalled (Photo 3).

There is a seep of approximately one gallon per minute at the juncture of the abutment with the bedrock outcrop 13.5 feet below the top of the abutment (Photo 4). At the time of inspection, the upstream water level was approximately 11.5 feet below the top of the dam, thus the seepage measured was under only about two feet of head.

Left Abutment - The approximately 60 foot long left abutment houses six sluice gates to the powerhouse canal. The concrete abutment has vertical upstream and downstream faces. The upstream face and the right end of the abutment are eroded (Photo 5) at an elevation most probably corresponding to the upstream water level when flashboards were in place. The downstream face is eroded to an elevation probably corresponding to former water levels in the canal, though not as badly as the upstream face (Photo 6).

c. Appurtenant Structures

Right Abutment Gates and Gatehouse - The three 4' x 4' low level sluice outlets in the right abutment at invert elevation 249.2 are controlled by floor stands inside the wood beam and corrugated metal gatehouse (Photo 4) atop the abutment. The manually operated floor stands appear operational, with one gate partially open. If the electric motors mounted beside the floor stands are still workable, a portable generator would be necessary to electrically operate the gates as there is not any electrical power source in the gatehouse. The easily accessable gatehouse has been heavily vandalized and is in a state of neglect and disrepair. In general, it presents a hazard to anyone who might happen to venture inside of it.

Left Abutment Gates and Gatehouse - A brick gatehouse built atop the 24 foot wide left abutment houses six worm gear sluice gate hoists, which presently appear to be non-operational. The gate hoists control flow through six 6' x 8' openings at invert elevation 254.7 to the powerhouse canal. At the time of inspection, one gate was partially open. Structurally, the gatehouse appears sound, however it has been vandalized (Photo 6).

Powerhouse Canal - An approximately 640 foot long canal, excavated from the old riverbank, extends from the left abutment gatehouse to the powerhouse at its downstream end. The heavily silted canal is confined on the left by a dry laid stone retaining wall and on the right by a concrete wall (Photo 7). The stone retaining wall is in good condition though slightly overgrown along the top with brush. The concrete wall extends from the main dam abutment to a section of the old riverbank which was left in place (Photo 13). The concrete canal wall then continues as a retaining wall along this portion of natural ground to the waste weir adjacent to the powerhouse at the downstream end of the canal. The concrete is in poor condition, exhibiting erosion (Photo 8). The natural ground between the canal and the river appears stable and is covered with a heavy growth of brush and trees, some of which are fairly large (Photos 13 & 14).

Canal Waste Weir - The 50 foot long waste weir at elevation 268.7 is a broad-crested concrete weir of trapezoidal cross-section at the right downstream end of the canal between the concrete canal wall and the concrete substructure of the powerhouse (Photo 10). The crest and downstream face were resurfaced in 1966 and reinforcing bars protrude from the downstream face of the weir. The upstream face of the weir is significantly spalled. A steel frame foot bridge spans the weir, however, no planking for the bridge is in place. There is a low level sluice through the weir abutment, the gate for which is presently in an open position and non-operational due to a broken hand wheel (Photo 11).

Powerhouse - The powerhouse, consisting of a brick superstructure atop a concrete substructure, appears to be structurally sound though weathered and vandalized. The powerhouse had housed two turbines which were each fed by a 14' x 9' opening in the upstream concrete bulkhead of the powerhouse. Flow through the turbines was controlled by timber slide gates with trash racks which are somewhat corroded. The slide gates appeared to be in a closed position, though a slight flow through the powerhouse was observed. The concrete bulkhead in the area of the slide gates is severely eroded (Photo 9). Along the riverbank downstream of the powerhouse is a granite gneiss masonry retaining wall which is in good condition, though slightly overgrown near its downstream end. The area where the powerhouse discharges back into the river is heavily silted (Photo 12).

d. Reservoir Area - An approximately 400 foot long concrete retaining wall along the right shoreline of the impoundment appears to stabilize the right shoreline. The wall itself is extensively spalled.

Upstream of the brick gatehouse at the left end of the dam is a 115 foot long curved retaining wall which is moderately spalled and slightly undermined. However there is no evidence of structural instability. The small bay created by the wall is heavily silted except for approximately 15 feet along the upstream wall of the dam abutment.

It is likely that some sedimentation directly upstream of the dam has occured.

e. Downstream channel - The boulder strewn channel downstream of the dam is broad and free of any obstructions which might impair the operations of the dam.

3.2 EVALUATION

Based upon the visual inspection, the dam is assessed as being generally in fair condition. The following features which could influence the future condition and/or stability of the dam were identified.

1. Though obscured by overflowing conditions at the site at the time of inspection, it appears that the spillway is extensively deteriorated, including horizontal cracking and possibly seepage along the downstream face of the spillway.

2. There is extensive deterioration of the concrete dam abutments, canal wall, powerhouse bulkhead and upstream retaining walls.

3. Through weathering and vandalism, the powerhouse and gatehouses have fallen into a state of disrepair. The gate hoists in the brick gatehouse at the left end of the dam, and the canal low level sluice gates are not operational. The gates at the right end of the dam are not operational, other than by manual means.

4. There is seepage at the juncture of the right dam abutment with the adjacent bedrock outcrop.

SECTION 4: OPERATIONAL PROCEDURES

4.1 REGULATING PROCEDURES

There are no regulating procedures followed for the dam at present.

4.2 MAINTENANCE OF DAM

Other than periodic attempts to secure the brick gatehouse and powerhouse against vandals breaking into them, the dam is not maintained at all. The owner performed one inspection of the facility on July 11, 1978.

4.3 MAINTENANCE OF OPERATING FACILITIES

There is no maintenance of the operating facilities presently performed

4.4 DESCRIPTION OF ANY FORMAL WARNING SYSTEM IN EFFECT

No formal warning system is in effect.

4.4 EVALUATION

The operation and maintenance procedures are generally poor with all areas requiring improvement. A formal program of operation and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a formal warning system should be developed and implemented within the time-frame indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. General - Collins Company Lower Dam is referred to as a run-of-river dam because the spillway spans the normal river channel and, during major floods, would be submerged by the tailwater.

b. Design Data - Water surface profiles for the river channel upstream and downstream of the Collins Company Lower Dam were obtained from 2 flood plain reports: 1) NED Army Corps of Engineers, "Flood Plain Information - West Branch and Farmington River, Canton, New Hartford, and Barkhamsted, Connecticut" dated May 1977, and 2) H.U.D. - F.I.A. "Flood Insurance Study - Town of Canton, Connecticut," Proof Copy, dated February, 1979. Peak inflow to both the upper and lower Collins Company dams due to the 1/2 PMF storm were considered to be the same. (See D-12 to D-17). The desired rating curves for flows up to the order of magnitude of the test flood (1/2 PMF) were obtained utilizing water surface profiles for the Collins Company Lower Dam as plotted on Appendix D-4.

c. Experience Data - The maximum flood at the site occurred during August 1955, when a peak outflow of 105,000 cfs overtopped the dam about 10 feet to elevation 287.

d. Visual Observations - No problem conditions were observed at the site which would affect the hydraulic performance of the facility.

e. Test Flood Analysis - The Collins Company Lower Dam watershed contains several lakes and reservoirs (see Section 1.3a) which could substantially reduce peak flows, especially when considering flows of a lesser magnitude than those due to a PMF storm. Considering the effect of these upstream reservoirs, it was determined that, while the reservoirs, with the exception of Colebrook, have very little reducing effect on peak inflows for a storm on the order of a PMF storm, there is considerable reduction of the peak inflow due to a 1/2 PMF storm (Appendix D-17).

The test flood for this significant hazard, small size dam is equivalent to one-half the Probable Maximum Flood (PMF). Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges", dated March, 1978, peak inflow to the reservoir is 83,000 cfs (Appendix D-1); peak outflow is 83,000 cfs with the dam overtopped 8 feet (Appendix D-7). Based upon our hydraulics computations, the spillway capacity is 33,000 cfs, which is approximately 40% of the routed test flood outflow. For this test flood, the spillway will operate under submerged conditions imposed by a tailwater stage to elevation 282, which is approximately 17 feet above the spillway crest and approximately 5 feet above the top of the dam.

f. Dam Failure Analysis - Two conditions for dam failure were analyzed to determine the hazard classification: 1) Failure of the dam with the water level at the top of the dam, and 2) Failure of the dam with the water level at the spillway crest. The peak failure outflow of 35,000 cfs from the dam breaching with the water level at the top of the dam would result in a 0.5 foot rise in the water level at the possible impact area, i.e., from elevation 272 to elevation 272.5 (D-11). An outflow of 33,000 cfs and tailwater stage to elevation 272 before dam failure would be sufficient in itself to cause evacuation of the possible impact area. Therefore, a breach of the dam causing a rise in the river level of 0.5 foot would cause no additional hazard in the downstream channel.

Utilizing the April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", a failure of the dam with the water level at the spillway crest elevation would result in a peak failure outflow of 7,900 cfs and a corresponding rise of 3 feet in the water level from elevation 254 immediately before the breach to elevation 257 immediately after the breach (D-11). A breach of the dam with the water level at the spillway crest would endanger the lives of persons downstream of the dam using the river for recreational purposes.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - Inspection of the dam and its appurtenances revealed extensive spalling accompanied by some cracking and erosion of concrete. There was no evidence, however, of immediate structural instability, although inspection of the 300 foot long spillway was not possible due to the concealing of the structure by overflow.

b. Design and Construction Data - There is not sufficient design and construction data to perform a complete, detailed stability analysis for the dam. Complete information should include information on the jointing of foundation bedrock, as well as information on actual uplift pressures and configurations of the foundation at the toe of the dam. A stability analysis presented by Edwin P. Ball on his "Detail Plan of Dam" dated June 1912, indicates a factor of safety for the dam stability of 2.6 assuming 9.0 feet of water over the spillway crest. A stability analysis was performed by DRC for the "Canton Hydroelectric Project, Feasibility Study" (Appendix B-32, 33). However, as the DRC did not have the essential information specified above to perform their analysis, their results may not be accurate. The dam has withstood major floods of up to 10 feet above the top of dam elevation, therefore it may be judged to be stable based primarily upon the visual inspection and its past performance.

c. Operating Records - The operating records do not include any indication of dam instability since its construction in 1912 and 1913, or since subsequent modifications were performed.

d. Post Construction Changes - It does not appear that there have been any post-construction changes to the dam which would adversely influence the stability of the structure.

e. Seismic Stability - The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the dam appears to be in fair condition. No evidence of immediate structural instability was observed, however the 300 foot long spillway across the Farmington River could be severely deteriorated. This could not be ascertained due to flow over the spillway concealing the downstream spillway surface.

Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, peak inflow to the reservoir is 83,000 cubic feet per second; peak outflow is 83,000 cubic feet per second with the dam overtopped 8 feet. Based upon our hydraulics computations, the spillway capacity is 33,000 cubic feet per second, which is equivalent to approximately 40% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that this assessment of the condition and stability of the dam is based upon the existing data, the visual inspection, past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within one year of the owner's receipt of this report.

d. Need for Additional Information - There is a need for more information as recommended in Section 7.2.

7.2 RECOMMENDATIONS

It is recommended that further studies, pertaining to the following items, be undertaken by a registered professional engineer qualified in dam design and inspection.

1. Inspection of the downstream face of the dam structure with the water level just below the spillway crest. The engineer should then make any necessary repair or renovation recommendations based upon his field observations of deterioration and/or seepage. Recommendations, made by the engineer, should be implemented by the owner.

2. If the degree of deterioration and seepage is severe, field investigations should be undertaken to compile the information necessary to perform a stability analysis. The engineer should then perform the analysis.

3. Both gatehouses and the powerhouse have fallen into a state of disrepair and should be repaired, or at least isolated from trespassers. All gates through the dam and its appurtenances should be made operable.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time frame indicated in Section 7.1.c and continued on a regular basis.

1. Round-the-clock surveillance should be provided by the owner during periods of unusually heavy precipitation and high project discharge. The owner should develop a downstream warning system to be used in case of an emergency at the dam.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. A program of inspection by a registered professional engineer qualified in dam inspection should be instituted on an annual basis. The inspections should be comprehensive and should include the operation of the low level outlet works.

4. Deteriorated concrete of the dam abutments, canal walls, canal waste weir, powerhouse bulkhead, and retaining walls upstream of the dam should be repaired. Planking for the steel frame foot bridge over the canal waste weir should be put in place.

5. The seep at the right concrete abutment interface with bedrock should be monitored for increases in volume or turbidity of flow.

7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT COLLINS COMPANY LOWER DAM

DATE: APRIL 26, 1979

TIME: 12:30 PM

WEATHER: OVERCAST, SHOWERS

W.S. ELEV. ____ U.S. ____ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. CALVIN GOLDSMITH	CG	CAHN ENGINEERS, INC.
2. PETER HEYNEN	PH	CAHN ENGINEERS, INC.
3. THEODORE STEVENS	TS	CAHN ENGINEERS, INC.
4. GONZALO CASTRO	GC	GEOTECHNICAL ENGINEERS, INC.
5. CHARLES OSGOOD	CO	GEOTECHNICAL ENGINEERS, INC.
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. CONCRETE DAM	ALL	
2. LEFT ABUTMENT GATE HOUSE	ALL	
3. POWERHOUSE CANAL	ALL	
4. POWERHOUSE	ALL	
5. CANAL WASTE WEIR	ALL	
6.		
7.		
8.		
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

Page A-2

PROJECT Collins Co. Lower Dam DATE 4-26-79

PROJECT FEATURE Concrete Dam BY ALL

AREA EVALUATED		CONDITION
~~DAM EMBANKMENT~~ CONCRETE DAM		
Crest Elevation		276.7
Current Pool Elevation		265±
Maximum Impoundment to Date		OVERTOPPED - AUG 1955
Surface Cracks		NONE OBSERVABLE - WATER PASSING OVER CREST
Pavement Condition		FAIR AT ABUTMENTS - NOT OBSERVABLE ALONG SPILLWAY
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		APPEARED GOOD
Horizontal Alignment		APPEARED GOOD
Condition at Abutment and at Concrete Structures		FAIR
Indications of Movement of Structural Items on Slopes		N/A
Trespassing on Slopes		N/A
Sloughing or Erosion of Slopes or Abutments		N/A
Rock Slope Protection-Riprap Failures		N/A
Unusual Movement or Cracking at or Near Toes		APPEARED TO BE HORIZONTAL CRACKING ALONG D/S FACE OF SPILLWAY
Unusual Embankment or Downstream Seepage		SEEPAGE AT RIGHT ABUTMENT - POSSIBLE SEEPAGE THROUGH SPILLWAY
Piping or Boils		N/A
Foundation Drainage Features		NONE OBSERVED
Toe Drains		NONE OBSERVED
Instrumentation System		NONE

PERIODIC INSPECTION CHECK LIST

Page A-3

PROJECT Collins Co. Lower Dam DATE April 26, 1979

PROJECT FEATURE Left Abutment Gatehouse BY All

AREA EVALUATED		CONDITION
OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE		
a) Approach Channel		
Slope Conditions		N/A
Bottom Conditions		SILTED IN TO ABOUT 15' FROM INTAKE
Rock Slides or Falls		N/A
Log Boom		NONE
Debris		SOME DEBRIS
Condition of Concrete Lining		RETAINING WALL - SPALLED
Drains or Weep Holes		NONE OBSERVED
b) Intake Structure		
Condition of Concrete		FAIR - SPALLED
Stop Logs and Slots		NONE OBSERVED

PERIODIC INSPECTION CHECK LIST

Page A-4

PROJECT COLLINS CO. LOWER DAM DATE APRIL 26, 1979

PROJECT FEATURE POWERHOUSE CANAL BY ALL

AREA EVALUATED		CONDITION
OUTLET WORKS-TRANSITION AND CONDUIT		
General Condition of Concrete		POOR - EROSION OF CANAL WALL
Rust or Staining on Concrete		NONE OBSERVED
Spalling		SOME
Erosion or Cavitation		EXTENSIVE EROSION
Cracking		MINOR
Alignment of Monoliths		N/A
Alignment of Joints		APPEARED GOOD
Numbering of Monoliths		N/A

PERIODIC INSPECTION CHECK LIST

Page A-5

PROJECT Collins Co. Lower Dam DATE April 26, 1979

PROJECT FEATURE Powerhouse BY All

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		
General Condition of Concrete		POOR
Rust or Staining		NONE OBSERVED
Spalling		MINOR
Erosion or Cavitation		YES - DEEP EROSION
Visible Reinforcing		NONE OBSERVED
Any Seepage or Efflorescence		NONE OBSERVED
Condition at Joints		FAIR
Drain Holes		YES D/S SIDE POWERHOUSE
Channel		POWERHOUSE TAILRACE
Loose Rock or Trees Overhanging Channel		NONE OBSERVED
Condition of Discharge Channel		SILTED IN

PERIODIC INSPECTION CHECK LIST

PROJECT Collins Co. Lower Dam DATE April 26, 1979

PROJECT FEATURE Canal Waste Weir BY All

AREA EVALUATED	CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a) Approach Channel	POWERHOUSE CANAL
General Condition	FAIR - DEBRIS AT OUTLET
Loose Rock Overhanging Channel	NONE
Trees Overhanging Channel	SOME - NO PROBLEM
Floor of Approach Channel	SILTED IN
b) Weir and Training Walls	
General Condition of Concrete	FAIR
Rust or Staining	MINOR
Spalling	YES U/S FACE WEIR & TRAINING WALL
Any Visible Reinforcing	NONE OBSERVED
Any Seepage of Efflorescence	NONE OBSERVED
Drain Holes	NONE OBSERVED
c) Discharge Channel	
General Condition	SILTED IN
Loose Rock Overhanging Channel	NONE
Trees Overhanging Channel	MINOR
Floor of Channel	SILT
Other Obstructions	NONE OBSERVED

APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE

COLLINS COMPANY LOWER DAM

EXISTING PLANS

A set of design drawings by Edwin P. Ball,
Engineer for the Collins Company,
Collinsville, Connecticut.

"Plan of Race Wall" April, 1911

"Plan of Bulkhead" Dec., 1911

"Plan of Bulkhead Gate and Frame" Dec., 1911

"Detail Plan of Sluice Gates for Dam and Canal" Dec., 1911

"Detail Plan of Bulkhead" Dec., 1911

"Plan of cut Stone for Powerhouse and Bulkhead"
(no date)*

"Cross Section of Power Plant" Dec., 1911

"Detail Plan of Dam" June, 1912

"Plan of Waste Weir" Mar., 1913

"Plan of Powerhouse" (eight sheets) May, 1913

"Plan of Dam and Bulkhead" Nov. 1913

Untitled survey and plan of dam, 1"=30' (No date)*

"Flash Boards - All Dams"
Cross Sections (B3028)
The Collins Company
Collinsville, Conn.
June 9, 1942

"Plan of Dam and Bulkhead"
The Collins Co.
Collinsville, Conn.
Dec. 20, 1956
(Tracing of E. P. Ball drawing probably from 1912)

"Layout of Flashboard Tie-Wires"
Lower Dam (B3038)
The Collins Co.
Collinsville, Conn.
Feb. 26, 1957

*Undated drawings arranged in assumed chronological order.

Cable Strength Computations"
Flashboards for Lower Dam (B2038)
The Collins Co.
Collinsville, Conn.
March 21, 1957

"Present 1'-0" Addition to Weir at Lower Power Plant"
The Collins Company
September, 1965
(two sheets)

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
Jan. 11, 1957	State Board for the Supervision of Dams State Office Building Hartford, Connecticut	G. F. Whitney Plant Engineer The Collins Company	Application for permit to install flashboards and data concerning typical high water conditions	B-4
June 19, 1963	Files	Water Resources Commission Supervision of Dams	Inventory Data	B-6
July 11, 1978	Victor F. Galgowski Supt. of Dam Maintenance Water Resources Unit	Paul Biscuti Civil Engineer Water Resources Unit	Dam Inspection Report	B-7
Dec. 26, 1978	Clarence Korhonen Development and Resources Corp.	Robert L. Nelson Engineering Geologist Foundation Sciences, Inc.	"Reconnaissance Engineering Geologic Investigation - Canton Hydroelectric Project"	B-8
May 3, 1979	Dean C. Porterfield Canton Conservation Commission	Mr. Clarence Korhonen Development and Resources Corp.	Stability and Stress Analysis Criteria and Summary (excerpt from Canton Hydroelectric Project Feasibility Study	B-28

CABLE ADDRESS "COLLINSAXE" HARTFORD CONN.
CODES
B C 5TH AND 6TH EDITION
LIEBERS. BENTLEYS
WESTERN UNION

COLLINS & CO.
HARTFORD
TRADE MARKS
LEGITIMUS

AXES. ADZE
HOES. MATTOCKS
HATCHETS MACHETES
PICKS. HAMMERS
CANE KNIVES. BRUSH HOOKS

THE COLLINS COMPANY
MANUFACTURERS OF
COLLINS EDGE TOOLS, MACHETES, AXES &C.

Collinsville, Conn. U.S.A.

January 11, 1957

State Board for the Supervision of Dams
State Office Building
Hartford 15, Connecticut

Attention: Mr. William S. Wise, Chairman

Dear Mr. Wise:

Following your letter of instruction of November 29, 1956, we are enclosing various photostats showing our lower dam on the Farmington River, gates, flash-boards, etc. We are applying for a permit to use five feet high controllable flash-boards. The flash-boards are constructed as follows:A Pipe is placed in a socket in the top of the dam and extended above the dam about six and one-half to seven feet. The lower five feet will be faced with one-inch pine boards in double thickness, which will be wired to the upright pins. From the top of the pins extending to points on the shore are wires or cables which may be reached from land and released as desired. Flash-boards of this height and this design have been in use since 1929, and to the best of our knowledge have caused damage neither to us or to anyone else.

Awaiting your favorable action, we are

Very truly yours,

THE COLLINS COMPANY

G. F. Whitney
Plant Engineer

encls.
GFW:gn

Drawings B-4059, B-3039, and B-4058
B-3028, B-4057
B-2033, B-3038

Typical High Water Conditions in the Farmington River at Collinsville, Connecticut

From January 1942 to January 1950 there was high water twenty-two (2 times. The date of the flood, amount of boards cut (deliberately) or ca away by water pressure, ice or debris, and height of water above the Lower Dam is tabulated below with some comments. The length of the Lower Dam is 300 feet and the normal height of the boards was 5.1 feet unless otherwise noted.

John E. Fletcher
February 8, 1957

Date	Boards Cut or Gave Way	Elevation	Comments
March 9, 1942	200'	6.7'	
March 17, 1942	none	5.6'	Only 3' boards on dam at this time because guy wire were not available.
November 25, 1942	none	8.1'	
December 30, 1942	110'	7.7'	There was about a 15 minut surge at this elevation du to ice jam letting go in Upper Pond. Maximum was 6. before this short duration surge.
May 26, 1943	none	6.5'	
November 9, 1943	none	6.3'	
March 7, 1944	none	6.0')	(These figures may be from
March 17, 1944	none	6.0')	(a few tenths to one foot
March 24, 1944	none	6.3')	(low because Bristol water
April 25, 1944	none	6.2')	(gauge was not functioning.
September 15, 1944	none	7.0'	Hurricane
January 1, 1945	85'	6.4'	Ice jam in Upper Pond let
March 18, 1945	none	7.0'	
April 26, 1945	60'	7.6'	
June 16, 1945	none	6.3'	
December 26, 1945	none	5.6'	
January 7, 1946	none	6.3'	
July 23, 1946	none	5.7'	
April 6, 1947	25'	7.0'	Debris made this hole afte boards had survived a cres one foot higher at 8'.
November 12, 1947	10'	7.2'	Debris
March 20, 1948	70'	7.2'	Initially 70' were lost an later another 30' went so that when the crest arrive on March 22nd, there was 100' gone.
March 22, 1948	100'	6.7'	
December 30, 1948	90'	6.9'	Cut at 6 P.M.
	140'	7.6')	(140' more gave way at 9 P.
December 31, 1948	230'	8.4')	(when ice jam in Upper Pond (let go making a total of (230' gone from this time (until flood crest arrived (between 4 and 5 P.M. Dec. (31st, some 19 hours later.

WATER RESOURCES COMMISSION
SUPERVISION OF DAMS
INVENTORY DATA

7

Inventoried By T. C.

Date 5-19-63

Collins Company CT-380

#220

Name of Dam or Pond [illegible]

Code No. F 39.2

Nearest Street Location [illegible]

Town [illegible] 212

U.S.G.S. Quad. [illegible]

Name of Stream [illegible] 72-55.6

Owner ST. OF CONN

Address [illegible] Lat 41-48.0

Pond Used For [illegible] RECREATION

Dimensions of Pond: Width ______ Length ______ Area 10A

Total Length of Dam 560' Length of Spillway 500'

Location of Spillway [illegible]

Height of Pond Above Stream Bed 15'

Height of Embankment Above Spillway 5'

Type of Spillway Construction [illegible]

Type of Dike Construction ______

Downstream Conditions [illegible]

:70

Summary of File Data [illegible] 5/1/57

[illegible]

Remarks [illegible]

[illegible]

[illegible]

[illegible]

Would Failure Cause Damage? YES Class B

B-6

★ THINK CASH! Send in a suggestion. You could win an award! ★
Send your suggestion to: *Employees' Suggestion Awards Program, 165 Capitol Ave., Hartford, 06115.*

Interdepartment Message

STO-201 REV. 3/77 STATE OF CONNECTICUT
(Stock No. 6938-051-01)

SAVE TIME: *Handwritten messages are acceptable.*
Use carbon if you really need a copy. If typewritten, ignore faint li

To	NAME: Victor F. Galgowski	TITLE: Supt. of Dam Maintenance	DATE: 11 July 1978
	AGENCY: Water Resources Unit	ADDRESS:	
From	NAME: Paul Biscuti	TITLE: Civil Engineer	TELEPHONE:
	AGENCY: Water Resources Unit	ADDRESS:	

SUBJECT

Collins Company Lower Dam

Pursuant to a request by Mike and Chuck for further inspection, I visited this site on Tuesday, July 11, 1978.

Major spauling of the concrete of the right abutment/ gatehouse exists possibly induced by major seepage along what appears to be a construction joint. The spauling has progressed to a point where large surface pockets exist on the downstream face (see photo).

Also, some seepage exists along the joint where the abutment joins a 20' high verticle bedrock outcrop.

It is impossible to detect whether a major crack runs the width of the abutment where the seepage is flowing, however, due to the massiveness of the concrete abutment above the area of spauling and the relatively low head generating flow through this area (approximately 15'), I do not believe there is any immediate danger of failure.

If maintenance measures are not taken, spauling of the concrete will continue, eventually becoming critical.

Paul Biscuti

PB:ljk

★

FOUNDATION SCIENCES, INC.

CABLE ADDRESS: FOUNSCIENCE
PORTLAND, OREGON

CASCADE BUILDING, PORTLAND, OREGON 97204
TEL. 503-224-4435

December 26, 1978

Development and Resources Corporation
455 Capitol Mall
Sacramento, CA 95814

Attention: Mr. Clarence Korhonen

Dear Mr. Korhonen

Enclosed for your use and distribution is one copy of each of our Final Reports entitled, "Reconnaissance Engineering Geologic Investigation, Phillips Hydroelectric Project, Croton Falls, New York" and "Reconnaissance Engineering Geologic Investigation, Canton Hydroelectric Project, Collinsville, Connecticut", dated December 26, 1978.

If you have any questions regarding our reports or require consultation, please do not hesitate to contact our office. We appreciate the opportunity to be of service to you on this project and the continued confidence you have in our services.

Very truly yours,

FOUNDATION SCIENCES, INC.

Robert L. Nelson

Robert L. Nelson
Certified Engineering Geologist (Oregon No. E502)

RLN:bh

Enclosures: 2 Final Reports
Quadrangle Report No. 16 (Canton Encl. No. 4)
Map (Canton Encl. No. 5)

INITIAL		ACTION	INFO	FILE
	JJS	☐	☐	☐
	[illegible]	☐	☐	☐
	[illegible]	☐	☐	☐
	[illegible]	☐	✓	
	[illegible]	☐	☐	
	[illegible]	☐		☐
	[illegible]	☐		
	[illegible]	☐	☐	
	RRB	☐	✓	
[illegible]	T/M	☐	☐	
		☐	☐	

RECONNAISSANCE ENGINEERING GEOLOGIC
INVESTIGATION

CANTON HYDROELECTRIC PROJECT
COLLINSVILLE, CONNECTICUT

FOR

DEVELOPMENT AND RESOURCES CORPORATION
SACRAMENTO, CALIFORNIA

TABLE OF CONTENTS

LIMITATIONS

This reconnaissance evaluation of the foundation conditions as related to the present adequacy or deficiency of the dams and appurtenant works is based on conditions which are mostly underground and cannot actually be seen, nor were they tested.

There is some historical information available on the design and construction of the dams, but no information on the original site investigation or their operational performance. It must be understood, therefore, that the conclusions and recommendations presented are based in large part on indirect and incomplete information about the actual foundation conditions, even to a much larger degree than if an adequate subsurface investigation had been performed. The information in this study is not a certification or guarantee of the present suitability of the existing structures for their intended purposes or of the foundation conditions of proposed structures.

I. Regional Geology

The Canton Hydroelectric Project is located in the crystalline uplands of western Connecticut, part of an extensive area of structurally complex metamorphic and igneous rocks known collectively as the Appalachian Highlands. The crystalline uplands represent rocks of sedimentary origin, possibly silty shales, sandstones and carbonates which have been highly folded and faulted. The geologic history of the area from the (Cambrian) sedimentary origin is complex and involves at least one major period of crustal deformation and associated metamorphism and igneous intrusion which occurred during the Acadian Orogeny (Middle and Late Devonian). This mountain building produced the folds and gneiss domes which are characteristic of the area. The time from the end of the Acadian Orogeny to the Triassic Period was a period characterized by more or less gradual elevation of the rocks with erosion and deposition over the central and possibly western portions of Connecticut. These sedimentary rocks were then faulted and tilted eastward. A portion of these red Triassic sediments lie just east of the project site along the fault contact with the underlying metamorphic rocks. After this period of deformation in the late Triassic Period, continued erosion reduced the area to one of relatively low relief, caused development of major stream valleys like the Connecticut and exposed the complex crystalline rocks formed during the earlier geologic history. These rocks, some of which are exposed along the stream bed of the Farmington River at the site, consist of schists, gneisses and intrusives including granitic, pegmatitic and ultramatic rocks.

II. Site Geology

Geomorphology

The maximum relief at the site from the river bed to the adjacent hills is about 400 feet with hillsides sloping at approximately 25° to 30°. The height of the river bank in the lower right side of the reservoir area is about 15 feet. On the left side of the lower reservoir the river bank rises to the maximum elevation of the adjacent hills. Slopes around the upper reservoir immediately adjacent to the shore are relatively flat with 5 to 10 feet of relief adjacent to the flood plain areas. The river has a gradient of about 1.5° in the project area and has a rocky bed with numerous bedrock outcrops.

Lithology and Structure

Material at the site consists of bedrock, natural river bed alluvium, alluvium deposited as a result of the dams, rip rap (and other bank protection) and colluvium from the adjacent hillsides. These materials in relation to the existing facilities are shown on Figure 1.

The exposed bedrock consists of medium hard to hard, gray, medium grained garnite - muscovite - biotite - quartz - feldspar schist and gneiss with lenses of amphibolite and graphite - mica - quartz gneiss.

The rock hardness terminology used is :

> medium hard -- can be picked with moderate blows of the geology hammer.
> hard -- cannot be picked with geology hammer but can be chipped with moderate blows of the hammer.

The attitude of the bedrock foliation (bedding) and major joints was measured at three locations; just downstream from the sluice house at the lower dam, at the vicinity of the power house at the upper dam and at the highway cut on Rt. 179 just south of Collinsville.

Table 1 summarizes these measurements.

TABLE 1

Lower Dam Area

Bedding		Joints Set 1		Joints Set 2		Joints Set 3	
Strike	Dip	Strike	Dip	Strike	Dip	Strike	Dip
337°	64° SW					306°	75° NE
353°	66° SW						
345°	56° SW						

Upper Dam Area

Bedding		Joints Set 1		Joints Set 2		Joints Set 3	
Strike	Dip	Strike	Dip	Strike	Dip	Strike	Dip
020°	69° NW	020°	38° SE	327°	59° NE	308°	54° NE
024°	79° NW	013°	68° SE				
027°	60° NW						
000°	37° W						

Highway Cut

Bedding		Joints Set 1		Joints Set 2		Joints Set 3	
Strike	Dip	Strike	Dip	Strike	Dip	Strike	Dip
005°	67° NW	028°	48° SE	358°	24° NE		
015°	71° NW	055°	52° SE	340°	16° SW		

The information in Table 1 indicates that the attitude of the bedding displays a general north-south strike and a relatively steep westerly dip. This orientation is determined by the Collinsville Dome which is the main structural feature in the area. The table also indicates that there are possibly three predominant joint sets. It was not possible to determine, with the time available for study, which were the major and minor sets. In general, the joints are tight and spaced moderately close (1' - 3').

The natural river bed alluvium exposed along the banks consists of sandy gravel and rounded cobbles. In addition, there are accumulations of silty to clean fine sand deposited on the inside of bends in the river between the upper and lower dam and above the upper dam on the left side of the reservoir, north of the old railroad bridge. Also, there appears to be sandy gravel and cobbles at the water's edge around most of the upper reservoir. It is likely that the fine sandy alluvium was deposited as a result of the dam construction.

It was not possible to observe the material deposited directly upstream of the two dams but it likely consists of saturated, possibly loose fine sand. This material presumably extends to the original bottom elevation of the reservoir adjacent to the upstream face of the dams.

The rip rap and other bank protection placed around the reservoir consists of subangular to rounded cobbles and boulders, stone walls constructed of quarry rock and concrete walls. Bedrock is exposed along large segments of the river bank between the upper and lower dams, forming natural shoreline protection.

The colluvium, primarily exposed on the left shore of the reservoir upstream from the lower dam, consists of micaceous silty sand with scattered cobbles and boulders. Bedrock probably occurs at a shallow depth beneath the colluvium.

III. SEISMICITY

Because of their similar regional geology and earthquake history, the Phillips and Canton sites will be considered together in the following discussion of seismicity. The earthquake history of the area was reviewed using current information from the National Geophysical and Solar-Terrestrial Data Center of the National Oceanic and Atmospheric Administration and is summarized on Figure 2. Figure 2 shows the location of all earthquakes with an intensity of V or greater which have occurred from 1643 to 1978 within a 150 kilometer radius at each site. Based on this data, there have been a total of 44 seismic events in the last 335 years.

Table 2 summarizes this data relative to the total number and approximate frequency of occurrence of earthquakes of each intensity.

TABLE 2 -- Earthquake Frequency

Maximum Intensity *	V	VI	VII	VIII
Total number of Earthquakes	33	5	4	2
Approximate Frequency of Occurrence	10/50 yrs.	2/50 yrs.	1/50 yrs.	1/100 yrs.

*Modified Mercalli Intensity Scale of 1931.

To obtain design parameters for assessing the performance of existing or proposed structures under seismic loading, it is customary to discuss two hypothetic earthquakes, namely the maximum probable and maximum credible earthquake. Although the definitions of these two terms and the method of assigning a value to each are not consistent in practice, they are generally described as follows.

The maximum probable earthquake is the intensity at the site from the strongest earthquake that has ever occurred. This event is considered to have a reasonable possibility of occurrence during the design life of the structure and is based on the earthquake history and geology of the area. All structures should be designed to remain functional during such an earthquake, although minor repairs may be required.

The maximum credible earthquake is the strongest earthquake that can be expected to ever occur at the site based on understandable mechanisms, such as movement along a nearby large fault. Generally, the primary use of the maximum credible earthquake is to check the capability of the dam to retain water without catastrophic structural failure. The dam crest may be displaced significantly, and control structures may be rendered inoperable as long as they do not rupture and result in total failure of the dam. Repairs may be major.

The maximum probable earthquake is considered to be an intensity VIII event occurring at a distance of about 40 kilometers from the site. This was an actual earthquake which occurred SE of the Canton site (see Figure 2) although it is not possible to tell which fault may have caused the earthquake.

The maximum credible earthquake is considered to be an event occurring along a 25 kilometer straight line segment of a fault just south of the Phillips site within 10 kilometers of the dam. Although no historic earthquakes are known to have occurred along this fault, it is considered the most critical fault for the purpose of this study. A fault with at least the same straight line segment length occurs just east of the Canton site.

Table 3 summarizes the data used for these two earthquakes and presents related parameters.

The maximum probable earthquake developed in this summary as indicated in Table 3 produces a maximum bedrock acceleration at the site of .075 g. This acceleration is consistent with the seismic risk map of the Uniform Building Code which places the sites in Zone 1 (minor damage).

Because of the proximity of seismic risk Zones 2 and 3 to the project sites (see Seismic Risk Map, U.B.C.), the maximum credible earthquake with a resulting maximum bedrock acceleration of .2 g as developed in this summary is not considered overly conservative.

TABLE 3

Earthquake Design Parameters

	Fault Length	Fault Distance	Earthquake Intensity *	Earthquake Intensity at Site *	Maximum Bedrock Acceleration at Site -g *
Maximum Probable Earthquake	?	40 (Kilometers)	VIII	VI	.075
Maximum Credible Earthquake	25 (Kilometers)	10 (Kilometers)	IX	IX	.20

*Earthquake intensities, bedrock accelerations and attenuations based on data developed by Seed, *Idriss* and Kiefer, Characteristics of Rock Motion During Earthquakes, 1969.

IV. FOUNDATION CONDITIONS

Observations

Upper Power House -- There appears to be no cracking of the brick walls or concrete foundation. The concrete foundation and training walls for the power house are in contact with bedrock on the downstream side of the structure. Bedrock outcrops also occur immediately upstream from the power house. The left training wall on the river side is in contact with bedrock. Some cracks are visible on the inside of the left training wall. Leaks occur at the contact of the training wall and bedrock and in the stone wall which serves as the right training wall. Overflow water from theforebay strikes the adjacent bridge pier with high velocity. The main forebay walls just upstream from the power house are constructed directly on bedrock. The rest of the forebay walls were submerged and their condition or construction could not be observed.

Lower Power House and Gate House -- There appears to be no cracking of the brick walls, concrete foundation or concrete outlet works. No bedrock is actually visible in direct contact with concrete foundations of these two structures, however.

Power Canal -- Minor irregular cracks and deterioration occur on the right wall of the power canal every 10-15 feet ±. Cracking and one inch ± of vertical separation of a joint occurs about 200' downstream from the power house where a slight bend in the wall was constructed. Most of the left side of the power canal is a quarry-rock wall (no motar).

Sluice House -- There appears to be no cracking of the concrete foundation. The concrete foundation, in direct contact with bedrock, is visible on the downstream wall. There are bedrock outcrops both up and downstream from the sluice house. Leaks occur between the bedrock and concrete foundation on the downstream wall. The bedrock cliff downstream from the sluice house is very damp. A concrete retaining wall extends upstream from the sluice house for a considerable distance. It shows no bulging or settlement near the sluice house. Above the wall, sloping up to the abandoned railroad bed, rocks and boulder rubble are exposed.

Lower Dam -- The crest appears straight (no bulging in downstream direction) and level (no sags when viewed from upstream). It was

not possible to examine the contact of the dam structure with the gate house or sluice house wall because of flowing water.

The even flow of water over the dam crest is disturbed by horizontal jets or sprays of water coming from the face of the dam. The sprays of water appear to be concentrated on the lower 1/3 of the dam face and arranged in continuous, somewhat irregular horizontal lines. No actual inspection at the concrete motar composing the dam could be made because of flowing water.

Upper Dam -- No bulging of the dam or settlement of the dam crest is apparent. No leakage appears to occur from between the stone blocks of the structure, however, water flowing over the crest prevented a more accurate determination. Bedrock is visible in direct contact with the stone blocks at each abutment and along most of the downstream toe of the dam. Some water was flowing from between the stone blocks and bedrock at the left abutment. Directly upstream from the right dam abutment for about 100 feet there is a sloping concrete slab which ajoins the highway bridge abutment. The shoreline upstream from the left dam abutment has rip rap for a considerable distance.

Bedrock -- Bedrock is exposed, in general, over the whole area downstream of the upper dam and in the proposed fish ladder location. Bedrock is not observed directly upstream of the dams except at the right abutment of the lower dam. Where bedrock is not exposed at the riverbed, it is expected to occur from 5 to 15 feet below the surface.

All of the schist and gneiss bedrock outcrops appear very hard and durable throughout the project area.

The strike of the bedding is oriented generally up and downstream or roughly perpendicular to the dam axses. The dip of the bedding is generally steep in a westerly direction. The strike of the joints is also generally perpendicular to the dam axses with the dip of the joint planes in a general upstream direction. The strike of the bedding and joints are generally parallel to portions of the forebay and canal walls which are oriented in a north-south direction. Joint and foliation planes intersect moderately frequently.

Reservoir Areas -- There was no evidence of slope movement or the potential for landsliding within the reservoir areas either between the upper and lower dams or upstream from the upper dam.

Old Railroad Bed -- From the lower dam to approximately 1500' upstream, the railroad bed appears to be constructed of rock rubble excavated from the nearby highway cut or is constructed directly on or very close to bedrock. The slope above the old railroad bed appears to be composed of large angular rocks excavated from the highway cut. From this point, to the old railroad bridge, the railroad bed becomes a slightly elevated embankment of sand and gravel.

V. CONCLUSIONS

Foundation Material

The foundation material beneath all the structures (dams, power houses, sluice house, forebays, power canals and etc) generally appears to have been of sufficient strength to support the loads imposed by these structures and other forces up to the present time. This is based on the fact that no settlement is detected along the dam crests. Also, no cracking is observed on any of the buildings. Most of the cracks on the right power canal wall, and on the training walls and foundations at the base of the upper power house and lower sluice house are likely related to erosion by water, or deterioration along joints and seams between successive concrete pours, and not to inadequate foundations. This conclusion is further supported by the hard and durable appearance of the bedrock throughout the area. Also, the available construction drawings indicate that the lower dam, together with the gate, power and sluice houses are founded on bedrock.

Regarding the apparent settlement in the right power canal wall, it is considered unlikely that poor foundation material has been the cause.

Although there are no drawings showing the upper dam foundation, it is considered very likely that the dam and appurtenant structures are all founded on bedrock. Drawings of the highway bridge, just downstream from the dam, indicate that the bridge footings are founded on hard bedrock. Also as mentioned previously, bedrock outcrops are extensive in the area.

Horizontal Movement

The attitude of the foliation and joints appears to present no adverse orientation which would cause horizontal movement of the dam or adjacent facilities along bedrock discontinuities. However, local variations in the attitude of these discontinuities are likely to occur. The effect of such variation on the stability of the bedrock foundation is impossible to assess without more detailed subsurface information.

Leakage

Significant leakage through the lower dam may be indicated by what appears to be horizontal jets or sprays coming from the

face of the dam. It is also possible that such an appearance could be caused by water flowing over the crest, striking a rough spot on the face and being deflected outward. Without close examination of these areas of apparent leakage it is not possible to determine if they are detrimental to the strength or stability of the dam. Other areas of leakage observed, appear to present no serious threat to the structures involved since the water is flowing out between non-erosive material. If water flowing through the dam was causing progressive erosion of the masonry concrete, serious structural problems, could, of course, result.

Uplift Pressures

Uplift pressures in excess of normal tailwater conditions could occur if there is a confined zone of seepage beneath the structures, either between the structure and the bedrock or through the bedrock foundation. It was not possible to observe the areas immediately downstream from the structures for indication of seepage. As a consequence, and without any peizometers to monitor, it is impossible to determine if uplift pressures exist. The near vertical orientation of many of the foliation and joint planes in the rock, however, may tend to drain sufficiently to prevent the buildup of excess hydrostatic pressure at the toe of the dam.

Potential Penstock Location on Railroad Bed

The abandoned railroad bed appears to be constructed of material which would provide an adequate penstock foundation (see previous description).

Slope Stability

There appears to be a very low potential for landsliding from seismic loading or other causes within the reservoir areas or at the dams and appurtenant structures.

Liquifaction

It is possible that the material deposited directly upstream of the dams could liquify during an earthquake. This would cause maximum lateral earth pressures to develop against the base of the dams from the liquified sand (together with the horizontal earthquake loading).

VI. RECOMMENDATIONS

Foundation

Before final assessment of the adequacy of the foundations, it is recommended to inspect those areas of the facilities which were either not visible or inaccessible at the time of this study. These areas include mainly the interior foundations of the power houses, gate house and sluice house, and the face of the dams, forebay walls and other areas which were covered by flowing water. (Possibly inspect during low flow.)

Leakage

If possible, before final assessment of the seepage or leakage conditions is made, the dams should be observed during periods when there is a full head but water is not flowing over the crest.

Excavation

Rock excavation techniques will be required in bedrock. It is very difficult to access the potential for damage to the existing structures from blasting without better knowledge of the particle velocity propagation characteristics of the site and integrity of nearby masonry concrete or stone block structures. Based on studies by Nicholls, Johnson and Duval ("Blasting Vibrations and Their Effects on Structures", Bureau of Mines Bulletin 656, 1971), a safe blasting limit based on a scaled distance* of 50 ft/lbs$^{\frac{1}{2}}$ may be used provided a particle velocity of 2.0 inches per second is not exceeded in the foundation soil and/or rock affected by the blasting.

Before any blasting is undertaken, however, it is recommended that samples of the concrete be obtained from nearby structures for evaluation of its condition and the extent of alkali-silica reaction which has taken place. In addition, the face of the stone block structures should be examined closely for evidence of horizontal movement at joints. Also, instrumented blasts should be conducted at the site to determine the particle velocity propagation characteristics. This is especially important if excavation for a fish ladder is required very close to existing structures (the dam structure and highway bridge, for example).

---

*Scaled distance is obtained by dividing the distance in feet by the square root of the charge weight per delay interval in pounds.

If excavation is made close to the base of existing foundations, great care must be exercised to avoid under-cutting foliation planes, joint planes or other rock defects which could cause failure of the over-lying material by slippage along the defect.

Because rock excavation near the base of the dam could create a high risk situation regarding structure stability, it is recommended to investigate fish ladder designs which do not require rock excavation. It is recommended, therefore, to perform an accurate topographic survey of the rock surface in the area involved. It may be possible then, to choose an alignment for the fish ladder which will provide the required entry elevation and location, and at the same time require no, or very limited rock excavation.

If rock excavation is necessary, it is recommended to orient the line drilling along the planes of foliation. The rock will split easier in this direction.

Stability Analyses

It is recommended to perform stability analyses of the dam structure under both the maximum probable and maximum credible seismic loading. These should include other extreme loading conditions such as: maximum hydrostatic head, water flowing over crest and lateral loading due to possible liquifaction of the sand which has accumulated against the upstream face of the dam.

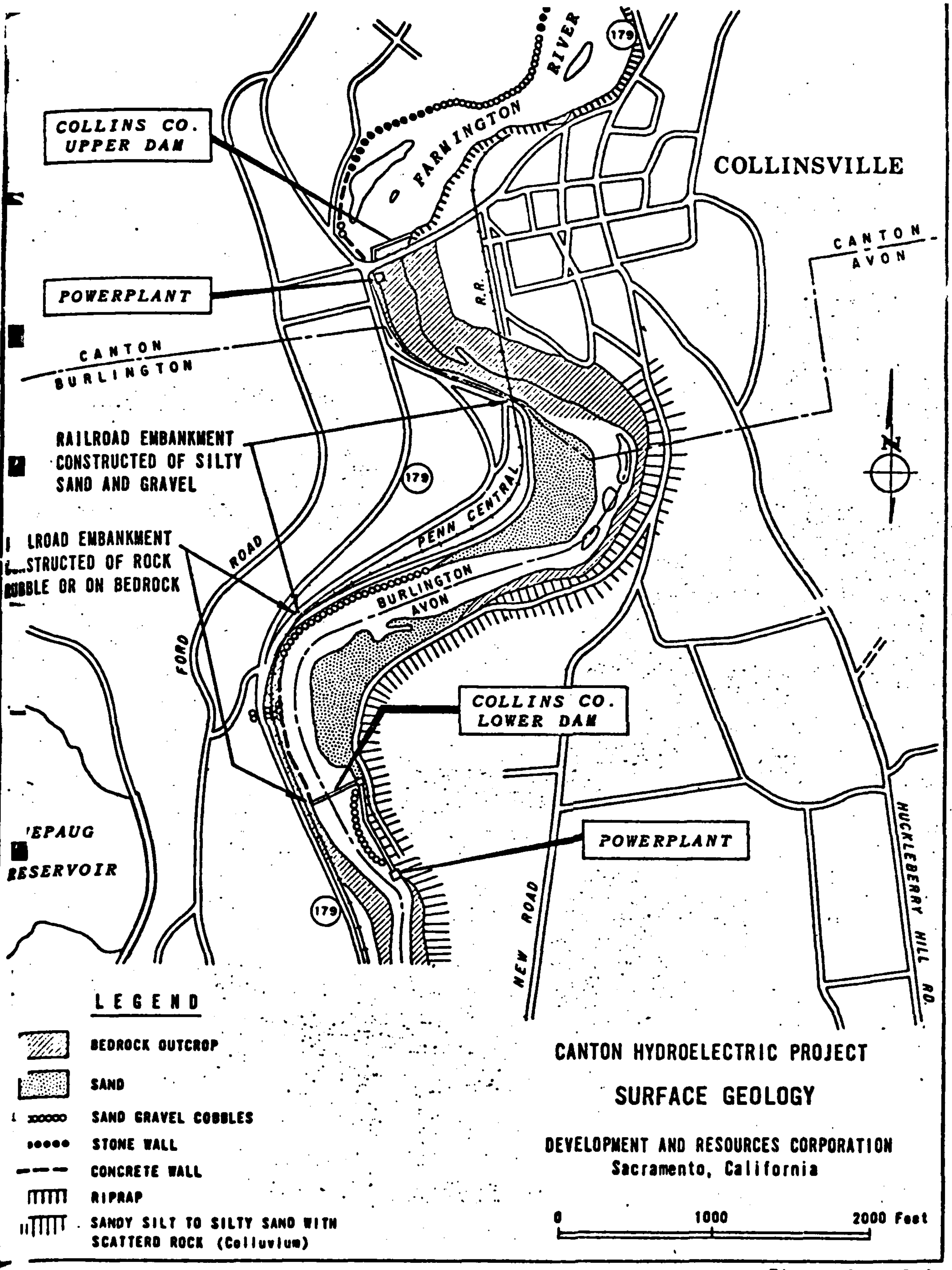
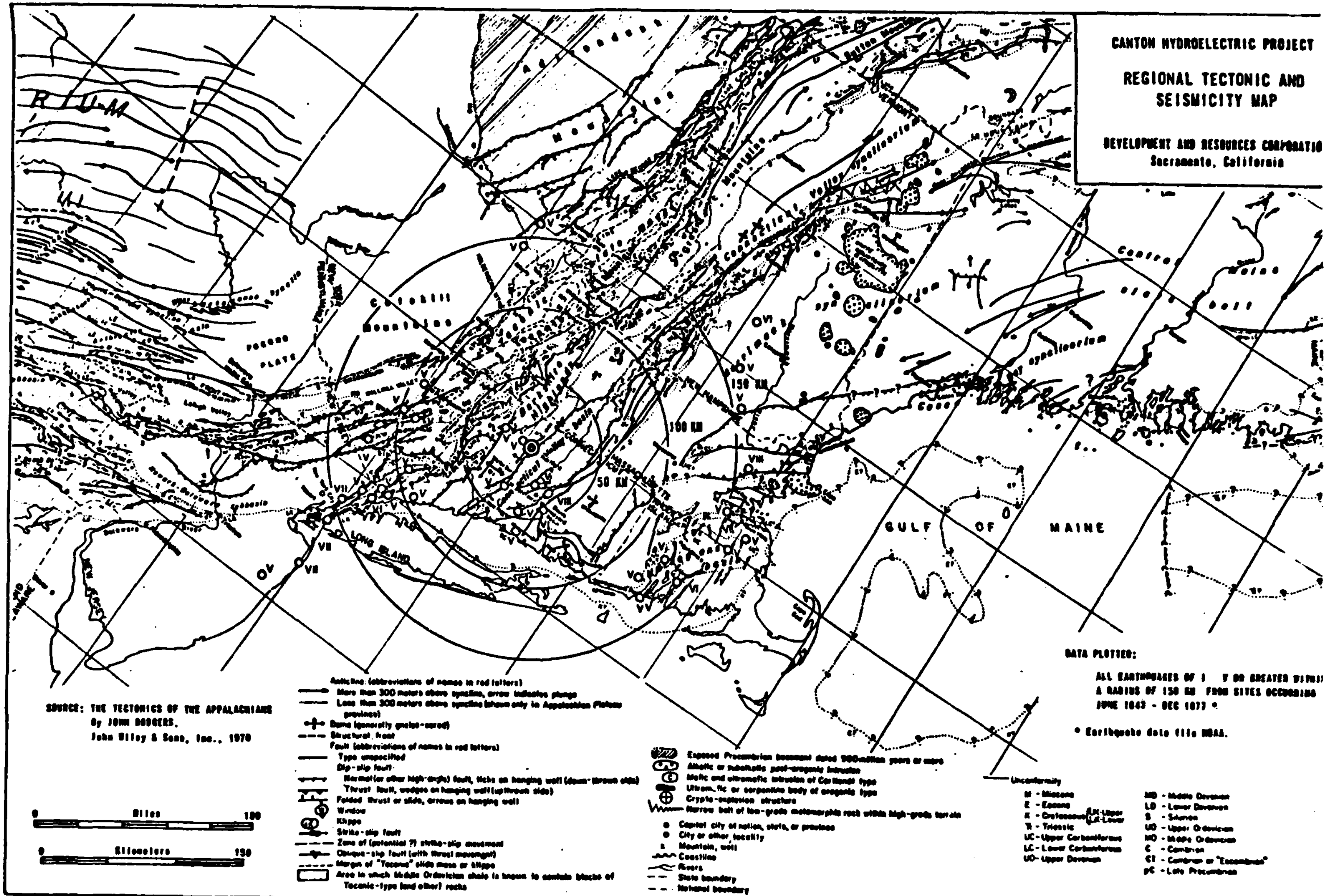


Figure App. B-1



CANTON HYDROELECTRIC PROJECT
REGIONAL TECTONIC AND SEISMICITY MAP
DEVELOPMENT AND RESOURCES CORPORATIO
Sacramento, California
GULF OF MAINE
LONG ISLAND
150 KM
100 KM
50 KM
DATA PLOTTED:
ALL EARTHQUAKES OF I V OR GREATER WITHI
A RADIUS OF 150 KM FROM SITES OCCURRING
JUNE 1643 - DEC 1977 *
* Earthquake data file NOAA.
SOURCE: THE TECTONICS OF THE APPALACHIANS
By JOHN RODGERS.
John Wiley & Sons, Inc., 1970
Miles
Kilometers
Anticline (abbreviations of names in red letters)
More than 300 meters above syncline, arrow indicates plunge
Less than 300 meters above syncline (shown only in Appalachian Plateau province)
Dome (generally gneiss-cored)
Structural front
Fault (abbreviations of names in red letters)
Type unspecified
Dip-slip fault
Normal (or other high-angle) fault, ticks on hanging wall (down-thrown side)
Thrust fault, wedges on hanging wall (upthrown side)
Folded thrust or slide, arrows on hanging wall
Window
Klippe
Strike-slip fault
Zone of (potential ?) strike-slip movement
Oblique-slip fault (with thrust movement)
Margin of "Taconic" slide mass or klippe
Area in which Middle Ordovician shale is known to contain blocks of Taconic-type (and other) rocks
Exposed Precambrian basement dated 900 million years or more
Mafic and ultramafic intrusion of Cortlandt type
Ultramafic or serpentine body of orogenic type
Crypto-explosion structure
Narrow belt of low-grade metamorphic rock within high-grade terrain
Capital city of nation, state, or province
City or other locality
Coastline
Rivers
State boundary
National boundary
Unconformity
M - Miocene
E - Eocene
UK-Upper
LK-Lower
Tr - Triassic
UC - Upper Carboniferous
LC - Lower Carboniferous
UD - Upper Devonian
MD - Middle Devonian
LD - Lower Devonian
S - Silurian
UO - Upper Ordovician
MO - Middle Ordovician
C - Cambrian
pC - Late Precambrian

Figure App. B

DIVERSION DAMS

Description and Condition

The Upper dam is approximately a maximum of 18 feet high and 350 feet long. This gravity overflow structure is composed of stone masonry with a vertical face on the downstream side. Steel pipes spaced at four feet have been installed at the crest of this structure to accommodate use of wooden flashboards up to 3.0 feet high. Visual inspection indicates that water passes through and between the wooden flashboards and, therefore, these units would need to be replaced for power generation. The dam itself, however, appears to be in good operating condition as no passage of water through the structure was noted and there have been no apparent lateral or vertical structure displacements. Plan drawings of the Collinsville Upper dam facility also indicate that the masonry structure is located directly in front of the original timber dam that was apparently left in place. No drawings or cross-sections of this older structure were available at the time of this study; and, it could not be visually inspected because of the river flows. The type and present condition of this timber structure could, therefore, not be assessed.

The Lower dam is a gravity overflow concrete structure approximately a maximum of 20 feet high with a crest length of 350 feet. During field reconnaissance, significant amounts of ravelling at the crest of this structure was indicated by the sharp jets and leakage of water passing over the crest. It should be further noted that the degree of deterioration at the crest is not known and that close examination of these areas would be recommended to determine the extent, if any, of leakage through the diversion structure. Progressive ravelling of the concrete caused by the passage of water through the structure could compromise the dam's structural integrity. No apparent vertical or horizontal structural displacements were noted during field inspections.

Dam Foundations

Visual inspection of the dam foundations at either the upper or lower sites could not be made because of flowing water. However, no lateral movement or settlement of the structures was noted during field

reconnaissance trips. Field inspection further indicates that there are many rock outcroppings between the upper and lower dams. Based upon the geological report on the area and visual observations, these rock formations are generally composed of schists and gneiss that are very hard and durable. Reference is made to the geology report included in Appendix B for a more complete description of the general regional and site geology.

An available detail drawing of the Lower dam indicates that this structure has been "keyed" into bedrock. These keys should prevent lateral displacement of the structure by the internal resistance of the key itself and the additional volume of foundation material that must be moved before the structure can slide. Furthermore, as judged by the strength of the surrounding rock formations, the structural capability of the foundation is considered to be competent and capable of withstanding the dam loadings and hydraulic flows to which it is subject.

The foundation for the Upper dam has been capable of sustaining the past dam and hydraulic loadings up to the present time. This is evidenced by the fact that no settlement or lateral movement of the dam could be noted during field reconnaissance trips. General surface geology report further indicates that there are many rock foundations in the vicinity of the Upper dam. Based on the Upper dam's past experience, coupled with the surface geology, it is felt that there is a strong possibility that the Upper dam is founded on firm hard bedrock which is capable of sustaining the required hydraulic and structure loads.

Stability Review

In order to assess the structural integrity of both diversion structures, analysis of each dam's structural loading conditions and stability were carried out. Calculations were based on the available section drawings and, for the purposes of calculation, each structure was considered to be

homogeneous in nature. Table II-1 displays both the loading conditions and the design criteria utilized for determining each of the dam's factors of safety with regard to stability.

The loading cases displayed in these tables represent the maximum loads that each dam would be subject to under normal, seismic, and flood conditions. In order to assess earthquake loading conditions, seismic events of two different intensities have been used as a basis for review. Thus, Case II has been defined as a probable earthquake intensity while Case III defines the maximum credible seismic event. In order to account for vertical earthquake accelerations, both the weight of water above the structure and the dam itself was modified by an acceleration factor equivalent to 50% of the horizontal seismic loads applied. Case IV represents the peak river discharges based on the 50-year flood condition.

In all load cases silt is assumed to be in place and is taken into consideration in determining the resultant loads to apply. This is because it is considered probable that over the years significant amounts of silt and sand have accumulated against the upstream faces of the dams. Since it is not known how impervious the silt or foundation may be, full hydrostatic heads are used as a measure of the uplift forces. Thus, a straight line variation from headwater to tailwater is used in evaluating the magnitude of uplift forces. It should be noted, however, that if the silt material deposited on the upstream face of the dams is clay-like, it could be relatively imperious. This would, therefore, change the flow path of water beneath the structures, creating a differential in uplift pressure across the dam which would be somcthing less than full hydrostatic. Since the actual differential in pressures is not known, both maximum and minimum possible uplift loads were utilized in the analysis of each diversion structure.

Based on the above loading conditions, factors of safety against overturning, uplift, actual sliding factors using stresses of each dam's base elevation were calculated. The results of these findings are displayed in Table II-2.

A possible problem with regard to stability could exist since calculations indicate that the dams' overturning factors of safety are below normally expected values. In view of these low factors, it is apparent that some type of anchorage at the toe of these structures most probably exists. The basis for this conclusion is also substantiated by the fact that both structures have withstood over 142 years and 65 years of flows respectively ranging to a maximum of at least 61,000 cubic feet per second (which occurred in the year 1955). This flow is approximately equivalent to a 250 year return frequency or a 0.4 percent chance of recurrence.

It is also possible that the bedrock which these structures are located on may tend to drain, thereby reducing the hydrostatic pressure and resulting uplift forces underneath the structures. It is recommended that the magnitude of pressures at the toe and heel of each structure be checked by field testing to determine the magnitude of actual uplift forces. Further review and structural analysis of each structure should then be carried out based upon the observed uplift pressures and actual anchorage conditions.

It is also necessary that a more detailed inspection of both Collinsville dams be made when the river flows can be diverted through the adjacent intake channels and/or sluice gates such that there is no water flowing over the crest of the dams. Such an inspection is required to verify that the downstream face of each structure is structurally intact and also to verify that there has been no undercutting at the downstream face at the interface with the bedrock. Signs of seepage should be looked for along with signs of deterioration of the cement mortar.
These activities would be included in the final site investigation and design stages of project implementation.

TABLE II-1

COLLINSVILLE DAMS

Design and Loading Criteria for Stability and Stress Analysis

Item	Design Loading Case			
	I	II	III	IV
Flashboards	Yes	Yes	Yes	No
Water Surface Elevation				
Upper	U/S=289.2 D/S=266.8	U/S=289.2 D/S=266.8	U/S=289.2 D/S=266.8	U/S=294.7 D/S=286.7
Lower	U/S=269.7 D/S=253.7	U/S=269.7 D/S=253.7	U/S=269.7 D/S=253.7	U/S=275.2 D/S=269.7
Reservoir Silting at Dam				
Upper	282.5=assumed existing level	282.5=assumed existing level	282.5=assumed existing level	282.5=assumed existing level
Lower	264.7=assumed existing level	264.7=assumed existing level	264.7=assumed existing level	264.7=assumed existing level
Uplift Pressure	100 percent	100 percent	100 percent	100 percent
Seismic				
Horizontal	0	0.075	0.20	0
Vertical	0	0.0375	0.10	0
Stability				
Sliding Factor	0.7	0.7	0.7	0.7
Water Pressure	62.4 pcf	62.4 pcf	62.4 pcf	62.4 pcf
Saturated Soil Pressure	86 pcf	86 pcf	86 pcf	86 pcf

STABILITY AND STRESS ANALYSIS SUMMARY

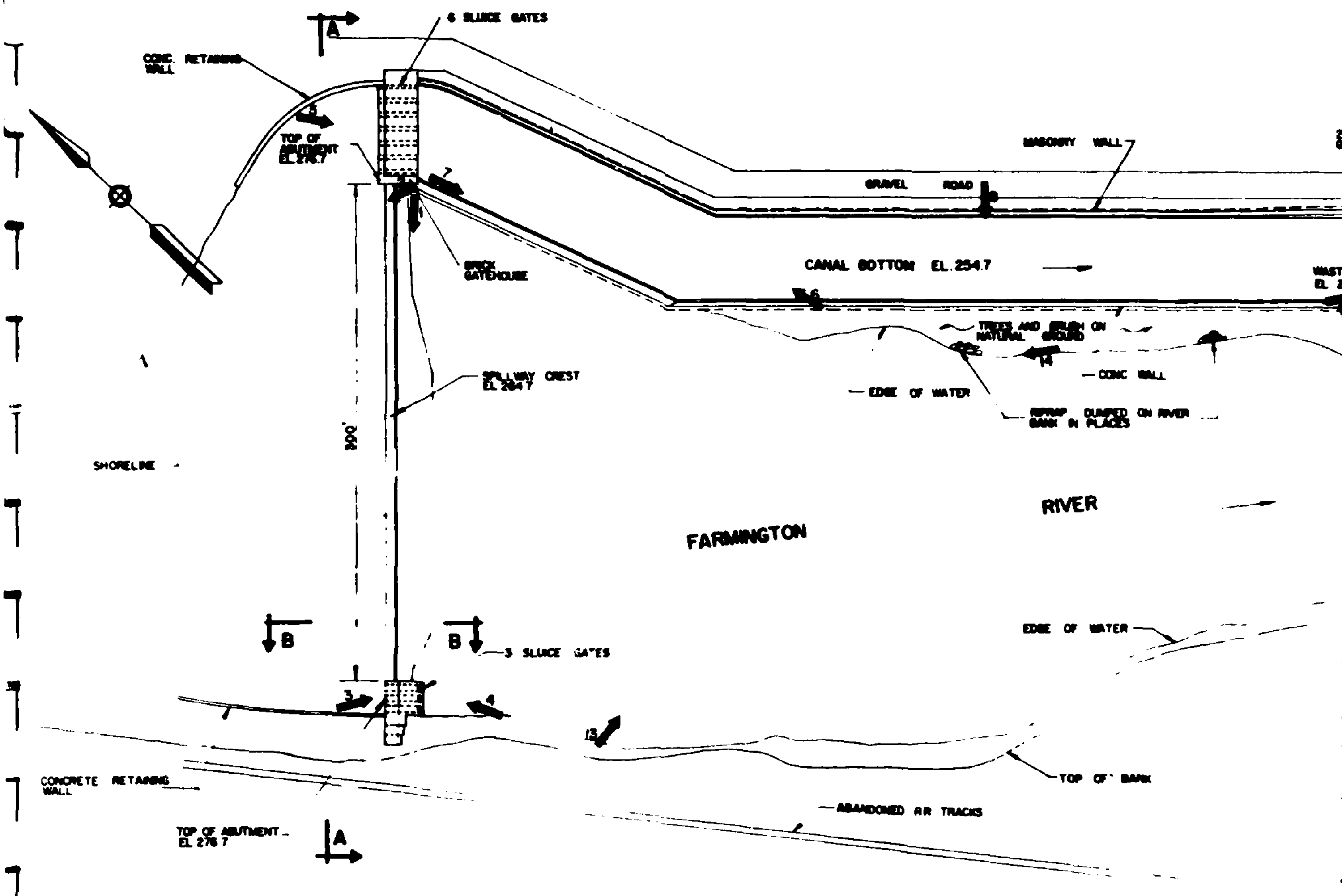
Item	Case Number			
	I	II	III	IV
LOWER DAM				
Stress (elevation 235.7)				
Heel (psi)	+24.8	+30.6	+40.2	+14.2
Toe (psi)	- 5.9	-13.2	-25.3	+ 7.4
Stability				
Uplift factor of safety	1.91	1.84	1.72	1.72
Overturning factor of safety with full uplift	1.21	1.06	.87	1.37
Overturning factor of safety without uplift	2.84	2.22	1.58	3.37
Sliding factor 2/	0	0	0	0
UPPER DAM				
Stress (elevation 267.83)				
Heel (psi)	+62.9	+69.9	+84.7	+44.5
Toe (psi)	-34.3	-42.7	-60.0	-25.6
Stability				
Uplift factor of safety	3.95	3.8	3.6	1.91
Overturning factor of safety with full uplift	.91	.76	.62	.93
Overturning factor of safety without uplift	1.32	1.04	.79	1.43
Sliding factor	.80	.99	1.36	.80
Actual sliding factor without uplift	.59	.73	.97	.38

All stresses and safety factors with full hydrostatic uplift forces unless noted otherwise.

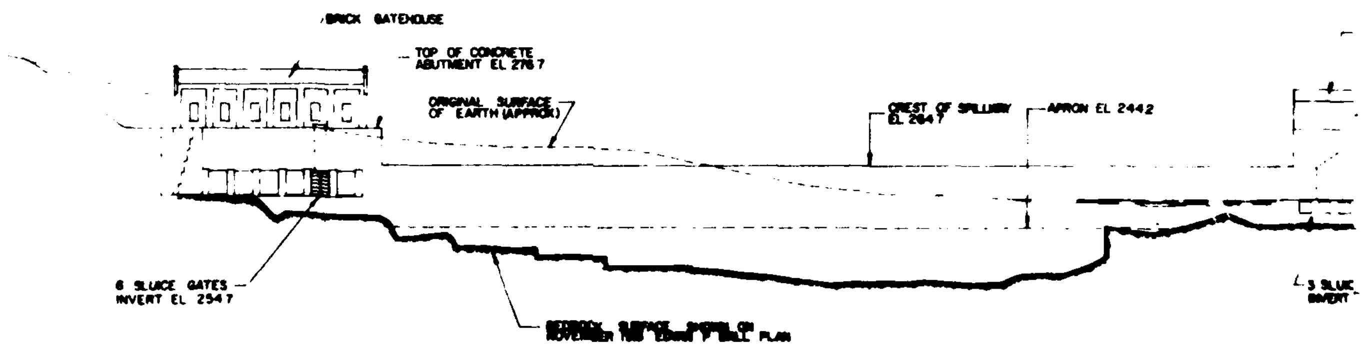
Lower dam keyed into bedrock which is assumed capable of resisting applied horizontal loads.

APPENDIX C

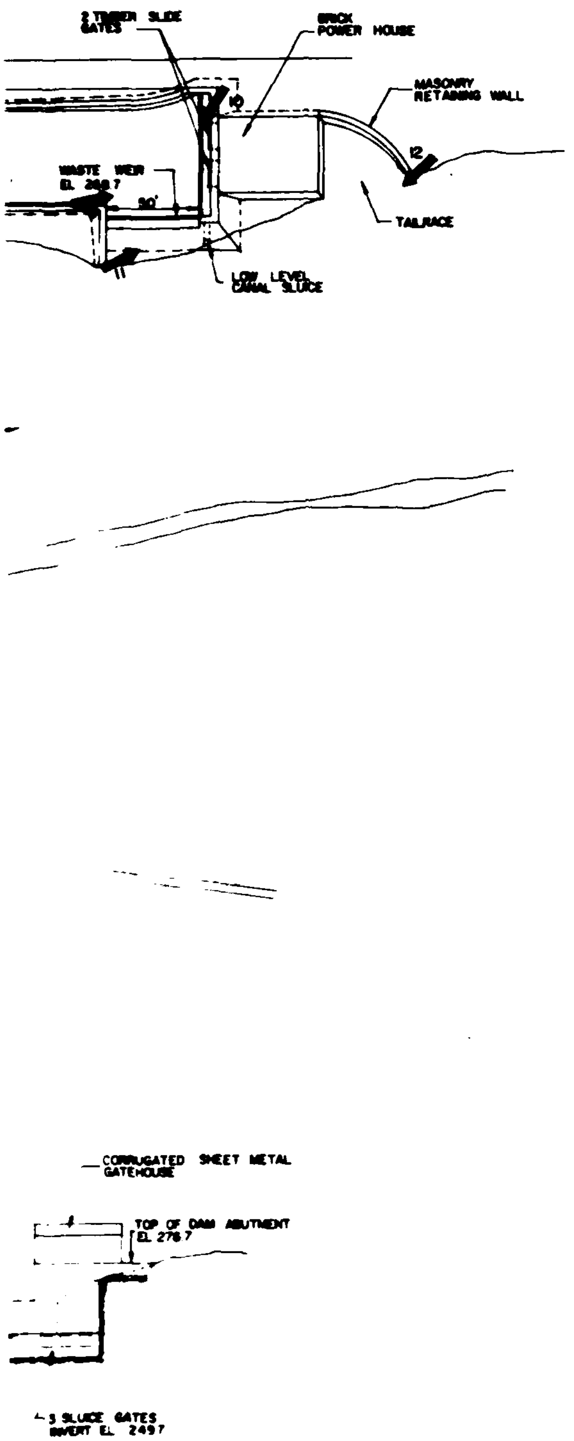
DETAIL PHOTOGRAPHS

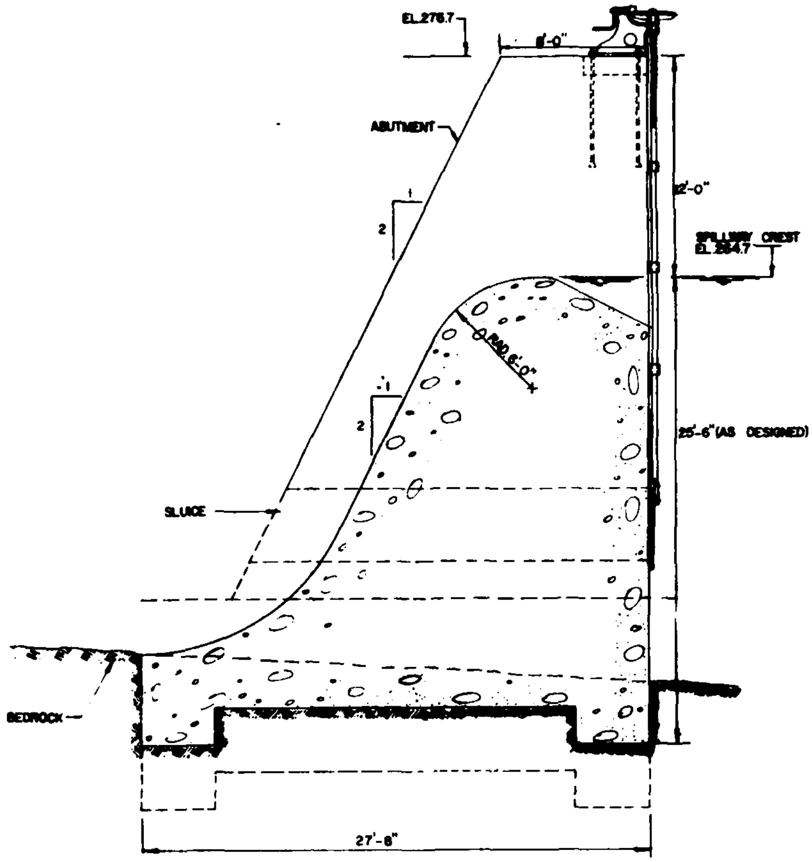


PLAN



SECTION A-A





SECTION B-B

5 0 5 10

(TAKEN FROM JUNE 1912 EDWIN.B BALL PLAN)

NOTES

1. THIS PLAN WAS COMPILED FROM EXISTING PLANS FOR THE DAM BY EDWIN P. BALL, ENGINEER.
2. ELEVATIONS SHOWN ARE MEAN SEA LEVEL DATUM. E P BALL DRAWINGS ARE ON COLLINS COMPANY DATUM. CONVERSION TO MSL GIVEN BELOW.

   COLLINS COMPANY DATUM + 186.2 = MEAN SEA LEVEL DATUM

3 ⬅ 2 PICTURE NUMBER AND DIRECTION

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION PLAN OF PHOTOS

COLLINS COMPANY LOWER DAM

FARMINGTON RIVER — AVON, BURLINGTON, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE: AS NOTED	
N.N.	TS	[illegible]	DATE: JULY 1979	SHEET C-1

(2)

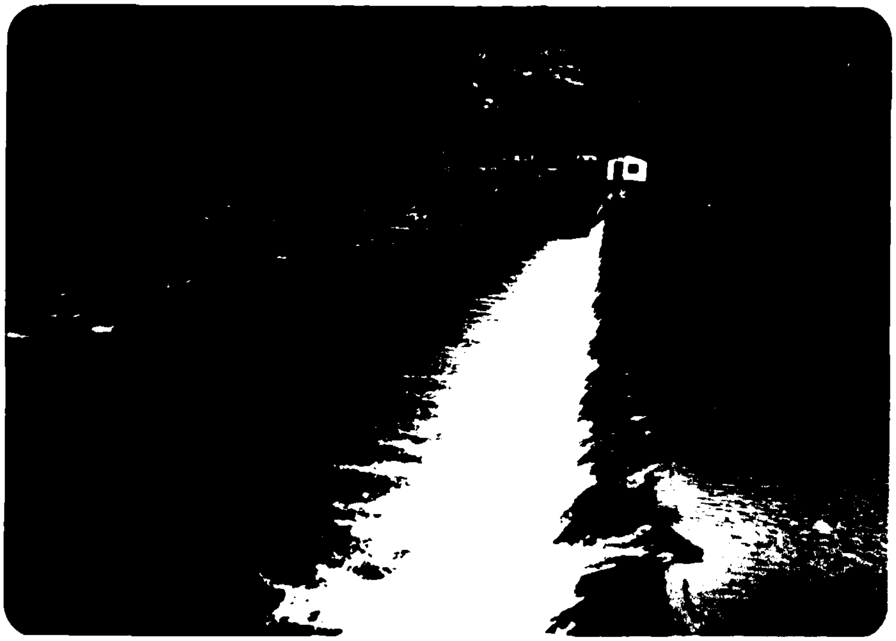


PHOTO 1 - Spillway crest and right abutment from left abutment. Note irregularity of crest (April, 1979).

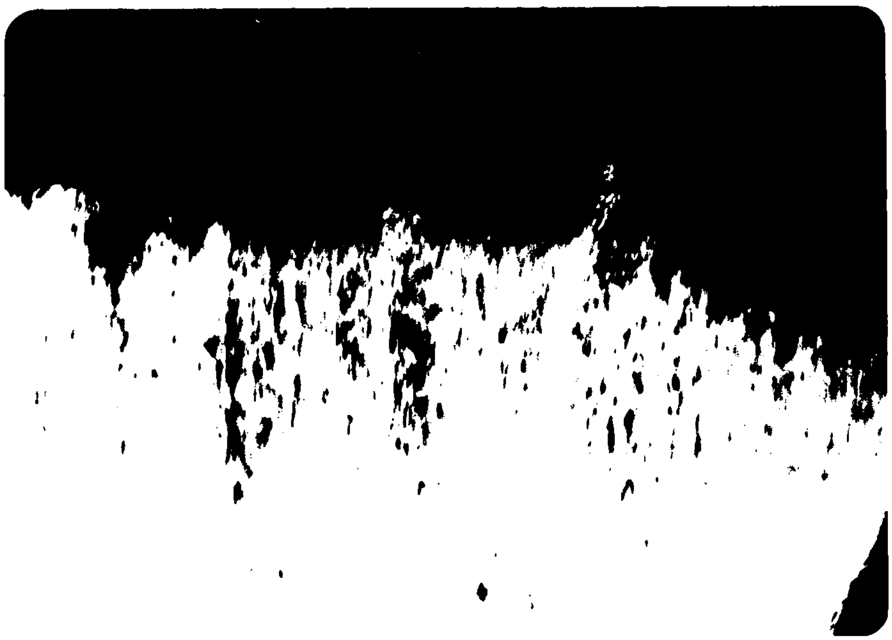


PHOTO 2 - Close-up of spillway crest showing deterioration (April, 1979).

US ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER		CE # 27 595 KB DATE July '79 PAGE C-1

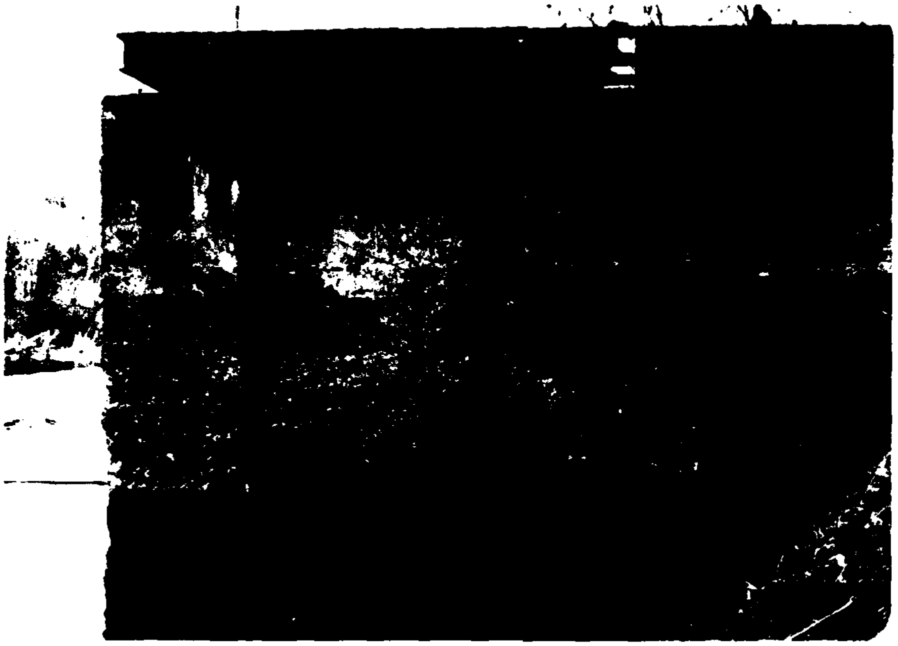


PHOTO 3 - Spalling of concrete on upstream face of right abutment. Note stems for gates (April, 1979).

PHOTO 4 - Spalling of downstream face of right abutment. Note seep at left natural ground contact; seep originates near base of small birch trees (April, 1979).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 595 KB DATE July '79 PAGE C-2

PHOTO 5 - Spalling of upstream face of left abutment (April, 1979).

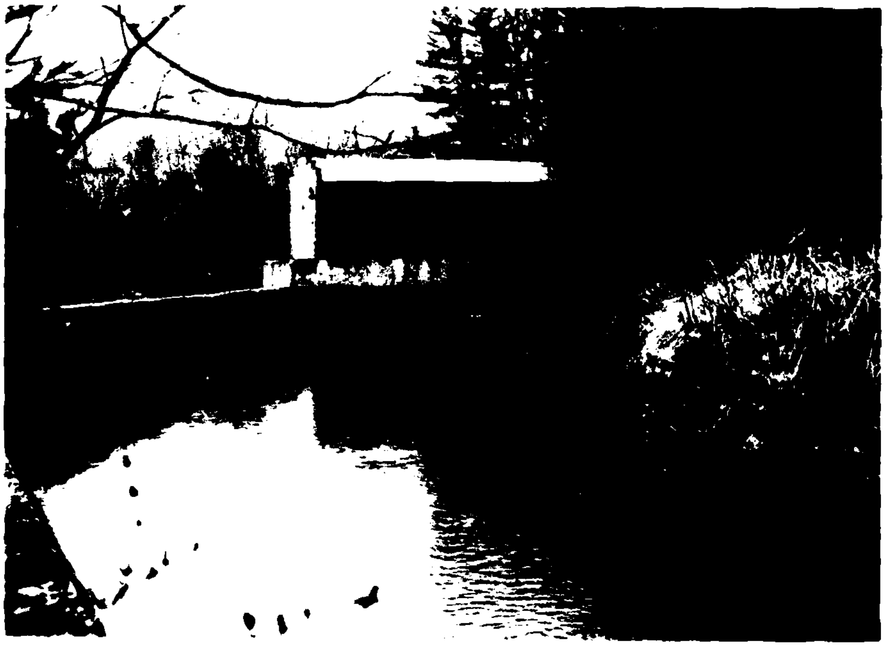


PHOTO 6 - Spalling of downstream face of left abutment at canal inlet. (April, 1979).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 595 KB DATE July '79 PAGE C-3

PHOTO 7 - View of canal to powerhouse from left abutment. Note canal siltation (April, 1979).

PHOTO 8 - Severe deterioration of concrete right canal wall (April, 1979).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER		CE # 27 595 KB DATE July '79 PAGE C-4

PHOTO 9 - Upstream face of powerhouse at downstream end of canal. Note floor stands upstream of powerhouse and lift to low level sluice at extreme right of photo (April, 1979)

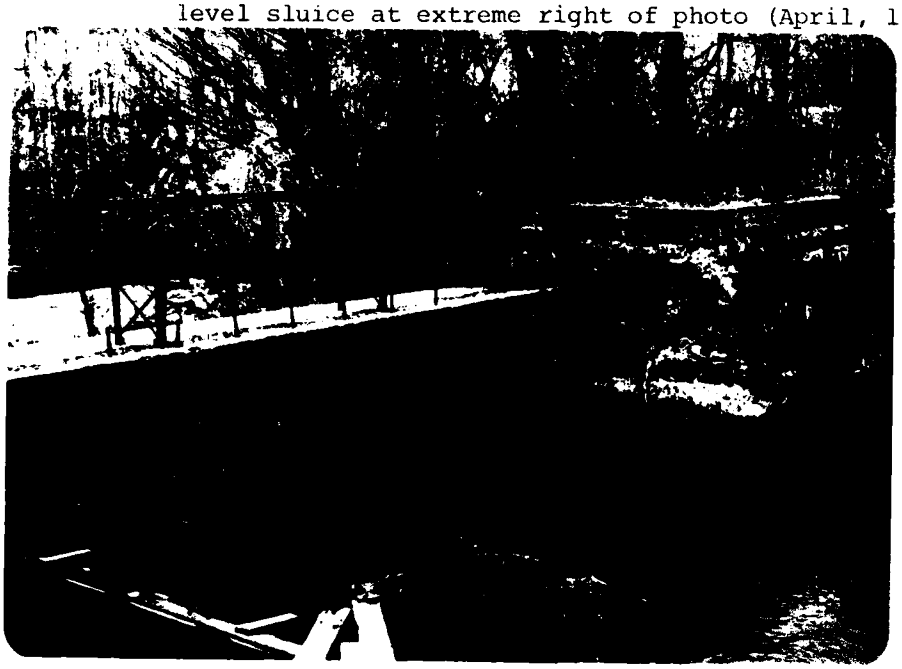


PHOTO 10 - Waste weir at downstream end of canal adjacent to powerhouse. Note spalling of concrete and debris in channel (April, 1979).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER		CE# 27 595 KB DATE July '79 PAGE C-5

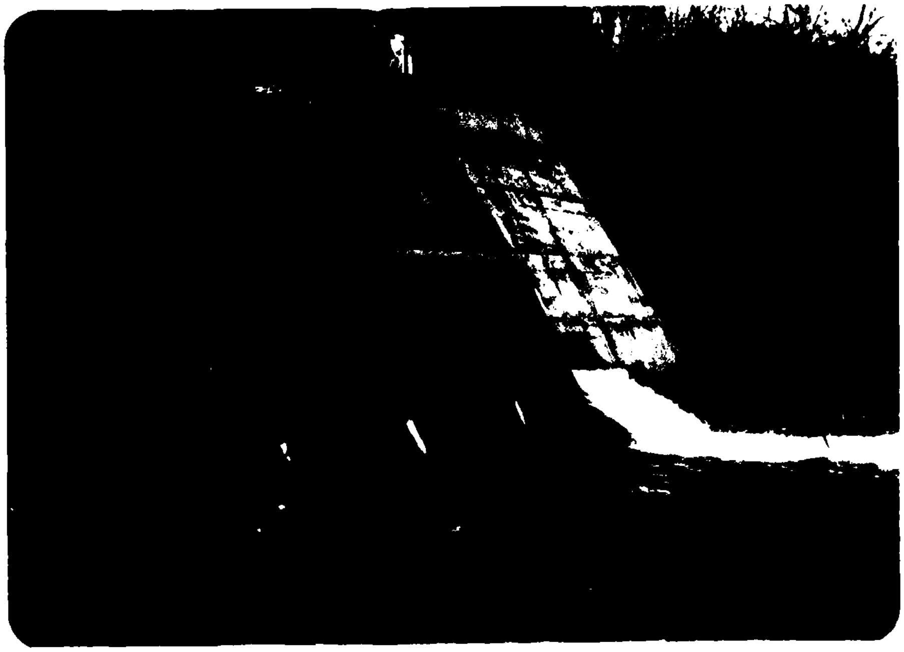


PHOTO 11 - View of toe of waste weir at powerhouse. Note sluice outlet (April, 1979).

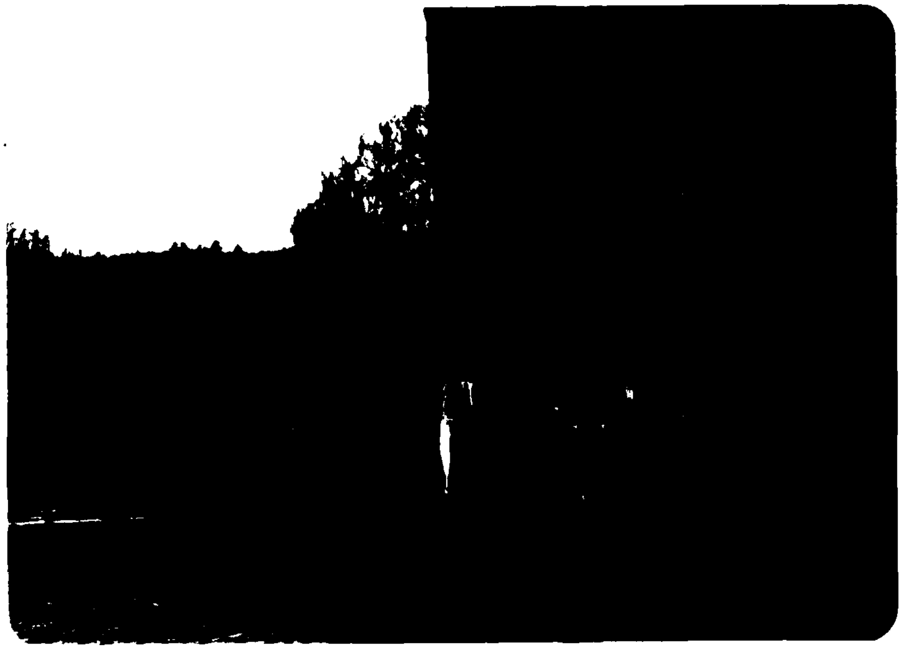


PHOTO 12 - View of powerhouse from downstream. Note waste weir abutment to left of powerhouse and siltation (April, 1979).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington. CT
CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER		CE # 27 595 KB DATE July '79 PAGE C-6

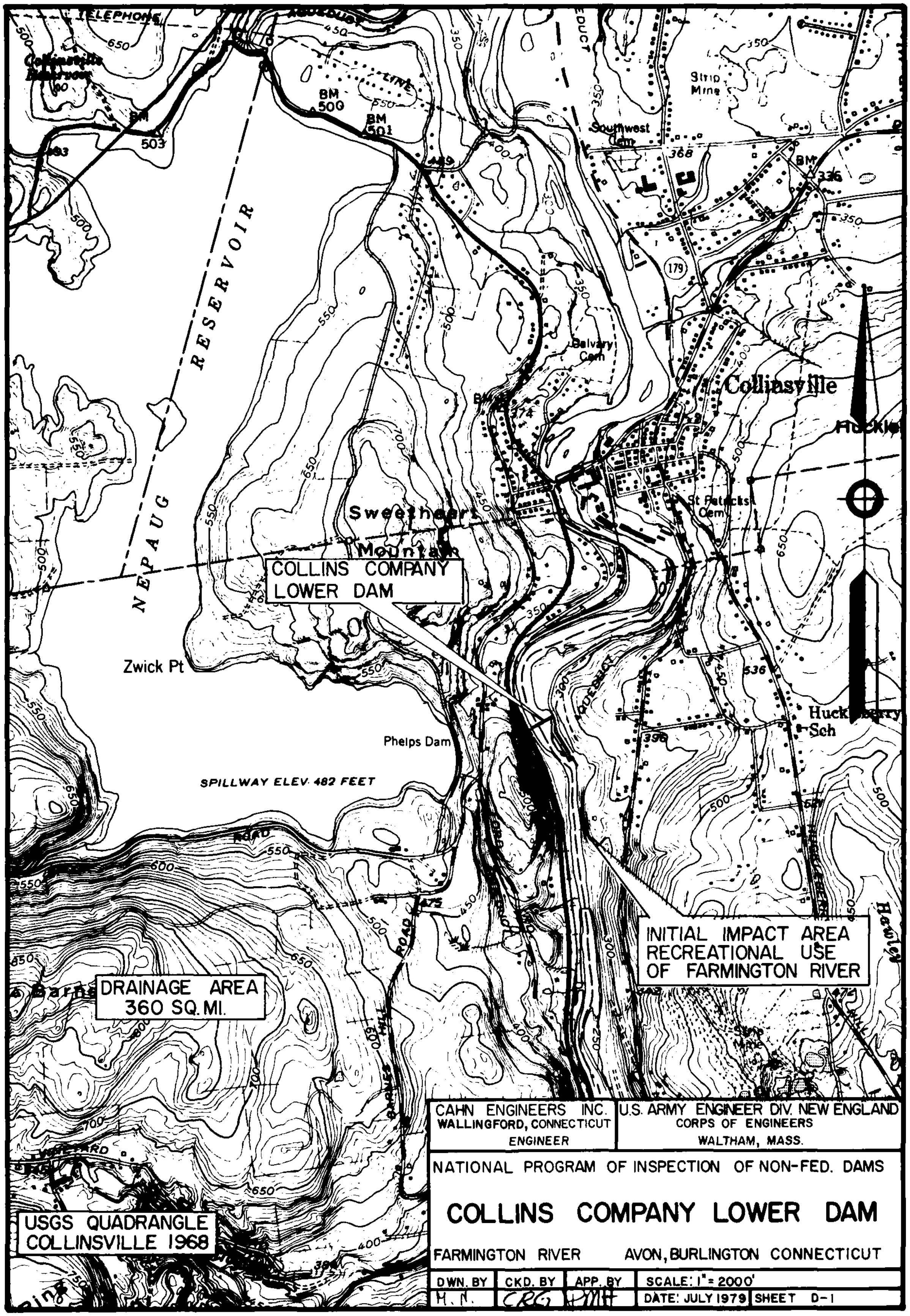
PHOTO 13 - View of downstream side of powerhouse canal. Note trees on slope after end of wall (April, 1979).

PHOTO 14 - Close-up of riprap and vegetation on downstream slope of powerhouse canal (April, 1979).

US ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Collins Co. Lower Dam Farmington River Avon-Burlington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER		CE # [illegible] DATE July '79 PAGE [illegible]

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS


NEPAUG RESERVOIR
Zwick Pt
Phelps Dam
SPILLWAY ELEV. 482 FEET
Sweetheart Mountain
COLLINS COMPANY LOWER DAM
Collinsville
Calvary Cem
Southwest Cem
Strip Mine
St Patricks Cem
Huckleberry Sch
INITIAL IMPACT AREA RECREATIONAL USE OF FARMINGTON RIVER
DRAINAGE AREA 360 SQ. MI.
USGS QUADRANGLE COLLINSVILLE 1968
CAHN ENGINEERS INC.
WALLINGFORD, CONNECTICUT
ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
COLLINS COMPANY LOWER DAM
FARMINGTON RIVER
AVON, BURLINGTON CONNECTICUT
DWN. BY H. N.
CKD. BY CRG
APP. BY
SCALE: 1" = 2000'
DATE: JULY 1979
SHEET D-1

Cahn Engineers Inc. Consulting Engineers

Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND
Sheet D-1 of 17
Computed By HM
Checked By TS
Date 7/22/79
Field Book Ref.
Other Refs. CE#27-595-KB
Revisions

HYDROLOGIC/HYDRAULIC INSPECTION

COLLINS CO. LOWER DAM, BURLINGTON/AVON, CT.

I) PERFORMANCE AT TEST FLOOD CONDITIONS:

1) MAXIMUM PROBABLE FLOOD

NOTE: COLLINS CO. LOWER DAM IS LOCATED (±) 1^{mi} D/S FROM THE COLLINS CO. UPPER DAM AND HAS A NOT MORE THAN $1^{sq mi}$ LARGER D.A. (DA ≅ $360^{sq mi}$). THEREFORE, PMF AND "½ PMF" FOR THESE TWO DAMS ARE FOR ALL PRACTICAL PURPOSES, THE SAME:

$$PMF = 215{,}000^{cfs}$$

$$\text{"}\tfrac{1}{2}PMF\text{"} = 83000^{cfs}$$

(SEE ATTACHED COLLINS CO. UPPER DAM H/H INSPECTION PMF COMPUTATIONS, DATED 7/16-17/79, pp. D-12 TO 17

2) SPILLWAY DESIGN FLOOD (SDF)

a) CLASSIFICATION OF DAM ACCORDING TO NED-ACE RECOMMENDED GUIDELINES:

i) SIZE: STORAGE (MAX) ≅ 690^{acft} ($50 < S < 1000^{acft}$)
HEIGHT ≅ 33' ($25 < H < 40^{FT}$)

STORAGE: ESTIMATE BASED ON THE FARMINGTON RIVER PROFILE FROM THE ACE - FLOOD PLAIN INFORMATION REPORT "WEST BRANCH AND FARMINGTON RIVER - CANTON, NEW HARTFORD AND BARKHAMSTED, CONN.", DATED MAY 1977 AND RIVER CROSS SECTIONS FROM AVAILABLE DATA ON SURVEY AND CONSTRUCTION DRAWINGS OF LOWER DAM (COLLINS CO., COLLINSVILLE, CT.-1911-1913); THE USGS

Cahn Engineers Inc. **Consulting Engineers**

Project NON-FEDERAL DAMS INSPECTION Sheet D-2 of 17
Computed By HCM Checked By TS Date 7/23/79
Field Book Ref. Other Refs. CE#27-595-KB Revisions

COLLINS CO. LOWER DAM

2,a - Cont'd) CLASSIFICATION

STORAGE (Cont'd) COLLINSVILLE QUAD. SHEET, 1968; AND, COLLINS CO. UPPER DAM / STATE HIGHWAY DEPT ROAD 765 DRAWINGS.

(1) STORAGE TO CREST OF SPILLWAY (ELEV. 264.7'MSL*), BY AVE. (TYPICAL) X-SECTION AND LONGITUDINAL RIVER PROFILE. $S \cong 160^{AC FT}$

(2) STORAGE TO TOP OF DAM (ELEV. 276.7'MSL*), BY THE SAME COMPUTATIONAL METHOD: $S_{max} \cong \underline{690}^{AC FT}$

NOTE: TYPICAL CHANNEL X-SECT. IS ASSUMED A TRAPEZOIDAL SECTION, 60' WIDE AT THE BASE WITH $1\frac{1}{2}^H$ TO 1^V AND 3^H TO 1^V SIDE SLOPES.

HEIGHT: FROM DATA ON COLLINS CO. DWGS No. 84057 AND No. 84058 AND SURVEY AND CONSTRUCTION DRAWINGS OF LOWER DAM. LOWEST D/S TOE ELEV. OF DAM CORRESPONDS APPROX. TO ELEV. OF APRON (EL. 244.2'MSL = 58'CCD) TOP OF DAM EL. 276.7'MSL. ∴ MAX HEIGHT OF DAM: $H \cong 32.5'$ SAY, $H = \underline{33'}$

ii) HAZARD POTENTIAL: NO LOW HOUSING OR OTHER PERMANENT STRUCTURES FOR HUMAN HABITATION WERE FOUND ALONG THE FARMINGTON RIVER D/S FROM COLLINS CO. LOWER DAM (APPARENTLY 5 LOW HOUSES (±) 2 mi D/S FROM THE DAM, SHOWN ON THE USGS COLLINSVILLE QUAD. SHEET, WERE SINCE THEN EITHER DESTROYED BY FLOODS AND/OR FIRE). THE FARMINGTON RIVER IN THIS AREA HAS A HIGH RECREATIONAL USE AND THEREFORE, PEOPLE MAY BE FREQUENTLY FOUND AT THE RIVER SIDE D/S FROM THE DAM.

*NOTE: COLLINS CO. UPPER AND LOWER DAMS DRAWINGS GIVE ELEVATIONS RELATIVE TO THE UPPER DAM SPILLWAY ELEV. 100' AS A DATUM (CCD). THUS, USCGS ELEVATIONS (MSL) = (CCD) + 186.2'

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet D-3 of 17
Computed By HOU
Checked By TS
Date 7/23/79
Field Book Ref.
Other Refs. CE#27-595-KB
Revisions

COLLINS CO. LOWER DAM

2-(Cont'd) SPILLWAY DESIGN FLOOD

iii) CLASSIFICATION:

SIZE: SMALL

HAZARD: SIGNIFICANT

NOTE: HAZARD CLASSIFICATION BASED ON THE HIGH RECREATIONAL USE OF THE FARMINGTON RIVER.

b) SDF = "½ PMF" ≐ 83000 CFS   PMF = 215000 CFS

3) SURCHARGE AT PEAK INFLOW

a) PEAK INFLOW: Q_p = 83000 CFS

b) SPILLWAY (OUTFLOW) RATING CURVE:

COLLINS CO. LOWER DAM IS A "RUN-OF-RIVER" DAM WHERE THE SPILLWAY LENGTH EQUALS APPROXIMATELY THE NORMAL STREAM WIDTH AND, ON MAJOR FLOODS, IT IS SUBMERGED BY THE TAILWATER. THE AUGUST 1955 FLOOD IS AN EXAMPLE OF THIS CONDITION.

THEREFORE, ESTIMATES OF SURCHARGE/SPILLWAY RATING CURVE FOR THIS TYPE OF DAM, PARTICULARLY FOR FLOODS OF THE ORDER OF MAGNITUDE OF THE TEST FLOOD, REQUIRE DETERMINATION OF THE TAILWATER WHICH SUBMERGES THE SPILLWAY (D/S CHANNEL RATING CURVE).

WATER SURFACE PROFILES FOR THE RIVER CHANNEL U/S AND D/S OF

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet D-4 of 17
Computed By [illegible] Checked By TS
Date 7/24/79
Field Book Ref. Other Refs. CE#27-595-KB
Revisions

COLLINS CO. LOWER DAM

3.b-Cont'd) OUTFLOW RATING CURVE

THE COLLINS CO. LOWER DAM THAT ALLOW DETERMINATION OF THE DESIRED RATING CURVES FOR FLOWS UP TO THE ORDER OF MAGNITUDE OF THE TEST FLOOD (Q_p) ARE AVAILABLE IN THE NED-ACE SUMMARY AND TECHNICAL REPORTS "FLOOD PLAIN INFORMATION - FARMINGTON RIVER - SIMSBURY, AVON AND FARMINGTON, CONNECTICUT", DATED MARCH, 1966.

THE U/S AND D/S W.S. ELEVATIONS FROM THE AVAILABLE PROFILES ARE USED AS PLOTTING POINTS FOR THE COLLINS CO. LOWER DAM SPILLWAY (OUTFLOW) RATING CURVE SHOWN ON NEXT PAGE (D-5).

A TABULATION OF THE SELECTED DATA IS AS FOLLOWS:

(1) DISCHARGE (Q-CFS)	U/S W.S. ELEV. (MSL)	D/S W.S. ELEV. (MSL)	
41000	279.5	274	
29000	275.5	270	
13000	270	262.5	
* 105000	287	285	* AUG. 1955 FLOOD HIGH WATERMKS.
** 61000	281.5	278	** AUG. 1955 FLOOD (MODIFIED BY U/S FLOOD CONTR. RESERVOIRS)
(2) 67000	282.5	279.5	

NOTES: (1) FROM CORRELATION BETWEEN ANNUAL PERCENT CHANCE OF FLOOD ON THE ABOVE MENTIONED REPORTS AND DATA ON NED-ACE - "FLOOD PLAIN INFORMATION - WEST BRANCH AND FARMINGTON RIVER - CANTON, NEW HARTFORD AND BARKHAMSTED, CT." - MAY 1977; "COLEBROOK RIVER DAM & RESERVOIR - DESIGN MEMO No 2 - HYDROLOGY, NOV. 1963 AND HUD-FIA "FLOOD INSURANCE STUDY - TOWN OF CANTON, CONN." PROOF COPY, FEB. 1979.

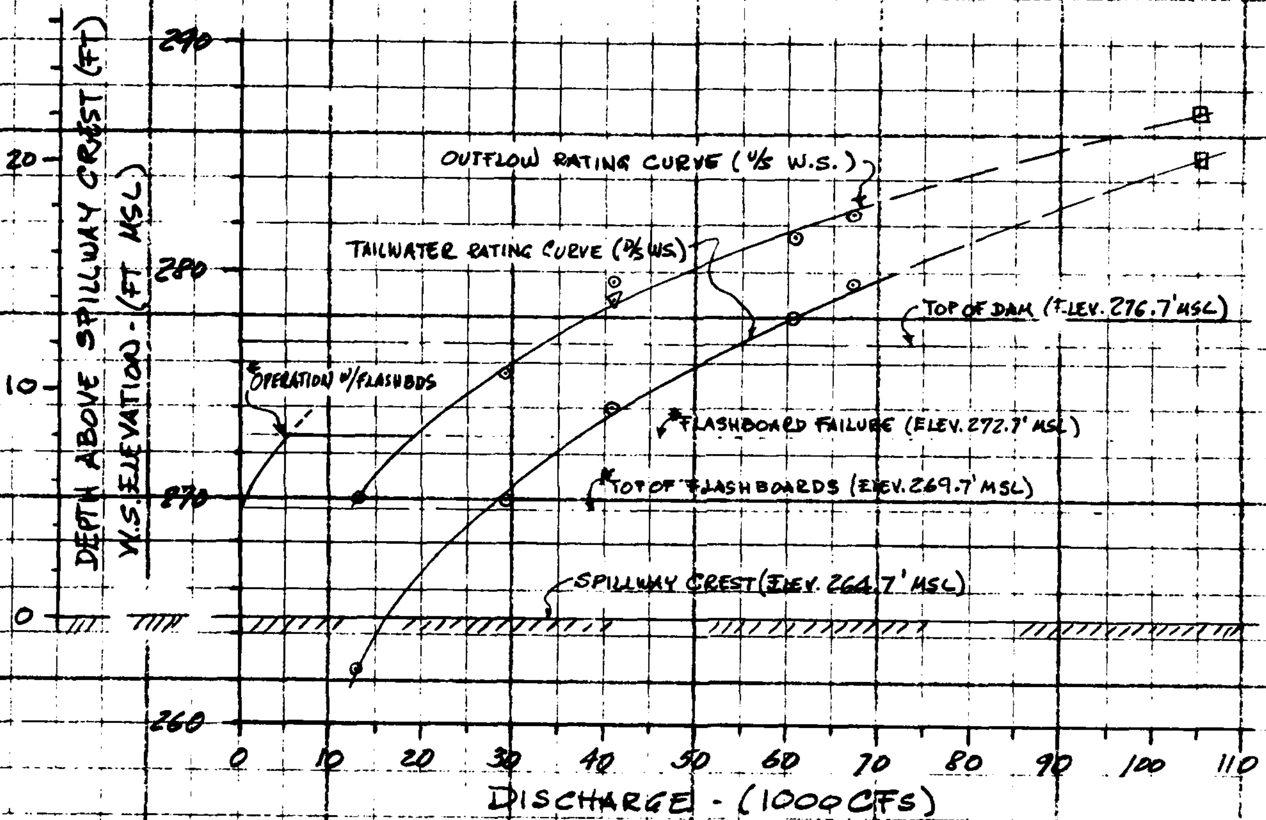
(2) SPF AT COLLINSVILLE FROM RATIO OF AUG 1955 FLOOD (MODIFIED AND UNMODIFIED) TO FLOODS AT RIVER GLEN, CT.

Cahn Engineers Inc. **Consulting Engineers**

Project: NON-FEDERAL DAMS INSPECTION — Sheet D-5 of 17
Computed By: [illegible] — Checked By: TS — Date: 7/24/79
Field Book Ref.: — Other Refs.: CE #27-595-KB — Revisions:

COLLINS CO. LOWER DAM

3.6 (Cont'd) OUTFLOW RATING CURVE



*NOTE: THE DAM HAS PROVISIONS FOR OPERATION WITH 5' FLASHBOARDS (NOT PRESENTLY INSTALLED) OVER THE ENTIRE LENGTH OF THE SPILLWAY (℄ L = 300'), DESIGNED TO FAIL AT A HEAD OF 2' TO 3'. THIS OPERATION WILL NOT HAVE ANY EFFECT ON CONDITIONS AT TEST FLOOD.

⊙ - ACE REPORTS
⊡ - AUG 1955 HWM
△ - C.E. ESTIMATE

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet D-6 of 17
Computed By [illegible]
Checked By TS
Date 7/25/79
Field Book Ref.
Other Refs. CE#27-595-KB
Revisions

COLLINS CO. LOWER DAM

3-Cont'd) SURCHARGE AT PEAK INFLOW

c) SPILLWAY CAPACITY TO TOP OF DAM

$H=12'$ (ELEV. 276.7' MSL) FROM CURVE, $Q_s \doteq 33000^{cfs}$

(($\pm$) 40% OF $Q_{p_1} = Q_{p_3}$)

SPILLWAY SUBMERGENCE BY TAILWATER OF ($\pm$) $H_{TW} \doteq 7'$

d) SURCHARGE HEIGHT TO PASS (Q_{p_1}):

Q_{p_1} = "½ PMF" $\doteq 83000^{cfs}$ ∴ $H_1 \doteq 20'$ SUBM.: $H_{TW} \doteq 17'$

4) EFFECT OF SURCHARGE STORAGE ON MAX. PROBABLE DISCHARGES (OUTFLOW):

a) PEAK OUTFLOW (Q_{p_3})

BECAUSE THE RESERVOIR STORAGE AT EXPECTED MAX. SURCHARGE (20') CORRESPONDS TO LESS THAN 0.1" R.O. OVER THE WATERSHED, NO APPRECIABLE PEAK REDUCTION IS PRODUCED AT THE RESERVOIR AND,

$Q_{p_3} \doteq Q_{p_1} \doteq 83000^{cfs}$ $H_3 \doteq 20'$ ($H_{TW} \doteq 17'$) FOR Q_{p_1} = "½ PMF"

b) SPILLWAY CAPACITY RATIO TO OUTFLOW:

THE SPILLWAY CAPACITY RATIO TO (Q_{p_3}) IS THEREFORE ($\pm$) THE SAME AS TO (Q_{p_1}) OR, ($\pm$) 40% (SEE 3, c ABOVE)

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet D-7 of 17
Computed By HM
Checked By TS
Date 7/25/79
Field Book Ref.
Other Refs. CE#27-595-KB
Revisions

COLLINS CO. LOWER DAM

6) SUMMARY:

a) PEAK INFLOW: Q_{p_1} = ½ PMF ≐ 83000 cfs

b) PEAK OUTFLOW: Q_{p_3} = Q_{p_1} ≐ 83000 cfs

c) SPILLWAY MAX. CAPACITY: Q_S ≐ 33000 cfs OR (±) 40% OF Q_{p_3} ≐ Q_{p_1}

THEREFORE, AT SDF = ½ PMF, THE DAM OVERTOPPED (±) 8' ((±) W.S. ELEV. 285' MSL) OR TO A SURCHARGE OF (±) 20' ABOVE THE SPILLWAY CREST ELEV. 264.7' MSL. THE SPILLWAY WILL OPERATE UNDER SUBMERGED CONDITIONS IMPOSED BY A TAILWATER STAGE AT (±) W.S. ELEV. 282' MSL OR (±) 17' ABOVE THE SPILLWAY CREST ((±) 5' ABOVE THE TOP OF THE DAM).

NOTE: BECAUSE CONDITIONS AT TEST FLOOD = PMF WILL REQUIRE ANALYSIS BEYOND A PHASE I INVESTIGATION AND EXTRAPOLATION OF THE OUTFLOW RATING CURVE ON P. D-5 IS NOT WARRANTED, PARALLEL COMPUTATIONS AT SDF = PMF WILL NOT BE MADE FOR THIS PROJECT.

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet D-8 of 17
Computed By [initials]
Checked By TS
Date 7/25/79
Field Book Ref.
Other Refs. CE #27-595-KB
Revisions

COLLINS CO. LOWER DAM

II) DOWNSTREAM FAILURE HAZARD

1) PEAK FLOOD AND STAGE AT IMMEDIATE IMPACT AREA:

a) FAILURE CONDITIONS OF DAM:

i) IF FAILURE IS ASSUMED TO OCCUR WHEN THE SURCHARGE IS TO THE TOP OF THE DAM, THE RIVER D/S FROM THE DAM IS AT A STAGE OF (±)272' MSL PRODUCED BY THE FLOW OVER THE SPILLWAY, JUST BEFORE FAILURE ($Q_S \cong 33000$ cfs. SEE P. D-6). TAILWATER IS SUBMERGING THE SPILLWAY AND THE SURCHARGE U/S FROM THE DAM IS (±)5' ABOVE THE TW. THEREFORE, UPON FAILURE OF THE DAM ON THESE CONDITIONS, A RISE IN THE STAGE OF MAX 2' TO 3' (SAY, TO W.S. ELEV. 275' MSL) IS EXPECTED D/S FROM THE DAM. (SEE NOTE ON P. D-11)

ii) FAILURE WHEN THE WL AT THE DAM IS AT SPILLWAY CREST ELEV. 264.7' MSL AND THE TAILWATER IS LOW, SAY,* W.S. EL. 254' MSL IS A CONDITION WHICH MAY RESULT IN CRITICAL FLOODING AND THEREFORE, WILL BE ANALYZED.

b) ANALYSIS FOR CONDITION (a, ii)

i) MID-HEIGHT (±) ELEV. 254.5' MSL ($\cong$ 68' CCD) (MID-HT. BELOW SPWY CREST EL. 264.7' MSL)

- SPILLWAY HEIGHT (SEE P. D-2) IS $H \cong 20.5'$

*NOTE: FROM NED-ACE TECHNICAL REPORT - "FLOOD PLAIN INFORMATION - FARMINGTON RIVER - SIMSBURY, AVON AND FARMINGTON, CONNECTICUT," DATED MARCH, 1966: "NORMAL WATER" W.S. EL. (±)255' MSL ∴ ASSUME LOW W.S. EL. = 254' MSL

Cahn Engineers Inc. **Consulting Engineers**

Project NON-FEDERAL DAMS INSPECTION Sheet D-9 of 17
Computed By HW Checked By TS Date 7/25/79
Field Book Ref. Other Refs. CE #27-595-KB Revisions

COLLINS CO. LOWER DAM

1.6 - Cont'd) - ANALYSIS FOR CONDITION (a, ii)

ii) APPROX. MID-HEIGHT LENGTH: $\ell \approx 230'$ (SCALED FROM COLLINS CO. LOWER DAM AVAILABLE DWGS.)

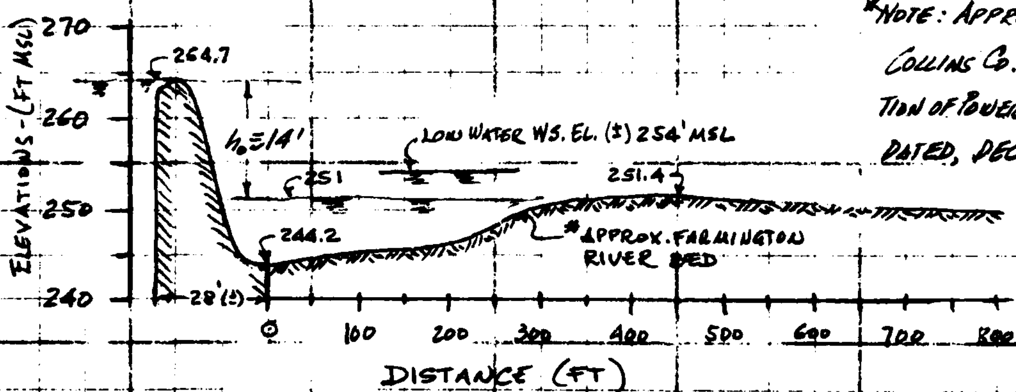
∴ iii) BREACH WIDTH (SEE NED-ACE D/S DAM FAILURE GUIDELINES):

$W = 0.4 \times 230 = 92'$ ∴ ASSUME $W_b = \underline{90'}$

iv) HEIGHT AT TIME OF FAILURE: $Y_o = 20.5'$

HOWEVER, BECAUSE OF THE D/S PROFILE OF THE RIVER BED (SEE SKETCH), THERE IS A PERMANENT POOL D/S FROM THE SPILLWAY WITH (±) W.S. ELEV. 251' MSL (65' CCD). THEREFORE, UPON FAILURE OF THE DAM, THE ACTUAL HEAD (h_o) WHICH WILL PRODUCE THE PEAK OUTFLOW WILL BE APPROXIMATELY:

$h_o \doteq \underline{14'}$



*NOTE: APPROX. PROFILE FROM COLLINS CO. DWG. "CROSS SECTION OF POWER PLANT," SCALE 1"=50', DATED, DEC. 1911 (No. 22).

v) BREACH OUTFLOW (Q_b):

$Q_b \doteq \frac{8}{27} W_b \sqrt{g}\, h_o^{3/2} \approx \underline{7900}$ CFS (EQUATION ALSO APPLIES TO THIS CASE)

COLLINS CO. LOWER DAM

1.b-Cont'd) - ANALYSIS FOR CONDITION

vi) PEAK FAILURE OUTFLOW:

$Q_{P_1} \doteq Q_b \doteq \underline{7900}^{cfs}$ (MINIMAL OR NO SPILLWAY DISCHARGE IS ASSUMED)

vii) FLOOD DEPTH (ABOVE POOL) IMMEDIATELY D/S FROM DAM:

$y \doteq 0.44 h_0 \doteq 6.2'$ SAY, $\underline{6'}$

viii) FLOOD STAGE IMMEDIATELY D/S FROM DAM: W.S. ELEV. $\doteq \underline{257}$' MSL

ix) RAISE IN STAGE AFTER FAILURE: $\Delta y \doteq 257 - 254 \doteq \underline{3'}$

2) SUMMARY:

a) FAILURE W/ SURCHARGE TO TOP OF DAM:

i) SPILLWAY (SUBMERGED) DISCHARGE
BEFORE FAILURE: $Q_s \doteq 33000^{cfs}$

ii) STAGE AT IMMEDIATE IMPACT AREA UPON FAILURE OF THE DAM, IS ESTIMATED TO RAISE MAX. 2' TO 3' OR, FROM (±) W.S. EL. 272' MSL TO (±) W.S. ELEV. 275' MSL. (SEE NOTE ON P. D-11)

b) FAILURE W/ SURCHARGE TO SPILLWAY CREST:

i) PEAK FAILURE OUTFLOW: $Q_{P_1} \doteq 7900^{cfs}$

ii) FLOOD DEPTH IMMEDIATELY D/S FROM DAM: $y \doteq 6'$ (ABOVE POOL W.L. (±) ELEV. 251')

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION Sheet D-11 of 17
Computed By HM Checked By TS Date 7/26/79
Field Book Ref. Other Refs. CE#27-595-KB Revisions

COLLINS CO. LOWER DAM

2.b-(Cont'd) SUMMARY - (FAILURE W/ SURCHARGE TO SPILLWAY CREST)

iii) APPROXIMATE STAGE D/S FROM DAM BEFORE FAILURE: W.S. EL. ≅ 254' MSL

iv) APPROXIMATE STAGE D/S FROM DAM AFTER FAILURE: W.S. EL. ≅ 257' MSL

v) RAISE IN STAGE AT ASSUMED IMPACT AREA: $\Delta y \cong 3'$

NOTE: A SIMILAR BREACH ANALYSIS TO THAT FOR CONDITION (i, ii) ON pp. D-8 TO D-10, GIVES FOR POOL AT TOP OF DAM A FAILURE DISCHARGE $Q_{P_1} \cong Q_s + Q_f \cong 35700$ cfs ($Q_f \cong 2000$ cfs, $h_o = 5'$). THEREFORE, THE CORRESPONDING RAISE IN STAGE MAY ACTUALLY BE ONLY (±) 0.5' OR, TO (±) ELEV. 272.5' MSL. (SEE p. D-5)

*ATTACHMENT TO COLLINS CO. LOWER DAM H/H INSPECTION.

HYDROLOGIC / HYDRAULIC INSPECTION

*COLLINS CO. UPPER DAM, CANTON, CT.

I) PERFORMANCE AT TEST FLOOD CONDITIONS:

1) MAXIMUM PROBABLE FLOOD

a) WATERSHED CLASSIFIED AS "ROLLING."

NOTE: THIS CLASSIFICATION IS ASSIGNED TO THE FARMINGTON RIVER WATERSHED AT COLLINSVILLE, FOLLOWING A GRADUAL CHANGE IN ITS D.A. CHARACTERISTICS FROM "MOUNTAINOUS" AT COLEBROOK DAM TO "MOUNTAINOUS TO ROLLING" AT NEW HARTFORD TO "ROLLING" AT COLLINSVILLE.

b) WATERSHED AREA

THE COLLINS CO. UPPER DAM WATERSHED CONTAINS SEVERAL LAKES/RESERVOIRS WHICH MAY SUBSTANTIALLY REDUCE PEAK FLOWS SPECIALLY THOSE OF LESSER MAGNITUDE THAN THE PMF. THEREFORE, THE POSSIBLE EFFECT THAT THESE RESERVOIRS MAY HAVE ON THE PEAK FLOWS WILL BE CONSIDERED IN THIS ANALYSIS.

i) TOTAL D.A. $\approx$ 359 sq mi (U.S.G.S. HARTFORD OFFICE)

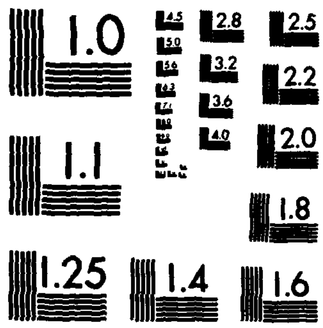
ii) D.A. OF WATERSHED REGULATED BY FLOOD CONTROL RESERVOIRS:

a.) COLEBROOK RES.: $(DA)_{CR} = 118$ sq mi (ACE DES. MEMO Nº 2, NOV. 1963)
b.) MAD RIVER RES.: $(DA)_{MR} = 18.2$ sq mi (ACE DES. MEMO Nº 1, OCT. 1960)
c.) SUCKER BROOK: $(DA)_{SB} = 3.43$ sq mi (ACE DES. MEMO Nº 1, JUN 1964)
(ii) TOTAL = 139.63

AD-A144 665 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS COLLINS COMPANY LOWER..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV JUL 79 2/2

UNCLASSIFIED F/G 13/13 NL

END



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet *D-13 of 17
Computed By [illegible] Checked By TS Date 7/16/79
Field Book Ref. Other Refs. CE#27-595-KB Revisions

*ATTACHMENT TO COLLINS CO. LOWER DAM P/H INSPECTION

*COLLINS CO. UPPER DAM

1,b-Cont'd) MAXIMUM PROBABLE FLOOD

iii) D.A. OF WATERSHED REGULATED BY OTHER RESERVOIRS:

a.) HIGHLAND LAKE (DIRECT D.A. D/S FROM SUCKER BROOK RES.) - $(DA)_{HL} = 3.54$ sq mi (ACE PH. I INSP.)
b.) SAVILLE (BARKHAMSTED) RESERV. - $(DA)_{SR} = 53.8$ sq mi (ACE PH. I INSP.)
c.) RICHARD'S CORNER (COMPENSATING) RES. (DIRECT D.A. D/S FROM SAVILLE RES.) - $(DA)_{RC} = 7.4$ sq mi (ACE PH. I INSP.)
d.) NEPAUG RESERVOIR $(DA)_{NR} = 31.9$ sq mi (ACE PH. I INSP.)
(iii) TOTAL = 96.64 sq mi

iv) UNREGULATED (DIRECT) D.A. TO COLLINS CO. UPPER RES.:

$$D.A. = 359 - (\underbrace{139.6 + 96.6}_{236.2}) = 122.8 \cong \underline{123} \text{ sq mi}$$

c) FROM NED-ACE "PRELIMINARY GUIDANCE FOR ESTIMATING MAXIMUM PROBABLE DISCHARGES" - GUIDE CURVE FOR PMF - PEAK FLOW RATES:

i) PMF (CSM) FOR THE TOTAL D.A.: $(PMF)_{T} \cong 600$ cfs/sq mi

ii) PMF (CSM) FOR THE UNREG. D.A: $(PMF)_{u} \cong 1200$ cfs/sq mi

d) PEAK INFLOW

PEAK FLOW REDUCTION BY THE VARIOUS RESERVOIRS IS TAKEN WHEN AVAILABLE, OR OTHERWISE ESTIMATED BY APPROXIMATE FLOOD ROUTING, FROM THE ACE DESIGN MEMOS AND/OR PHASE I INSPECTION REPORTS FOR THE RESPECTIVE RESERVOIR. THE FLOOD CONTROL RESERVOIRS ARE ASSUMED EMPTY AT THE BEGINNING OF THE TEST FLOOD - (NED-ACE).
AN ACCOUNT OF THE OUTFLOW FROM THESE RESERVOIRS AND D.A. CONTRIBUTING TO

*ATTACHMENT TO COLLINS CO. LOWER DAM H/H INSPECTION

*COLLINS CO. UPPER DAM

1,d - Cont'd) PEAK INFLOW

THE PEAK AT COLLINSVILLE (SITE OF THE COLLINS CO. UPPER AND LOWER DAMS), FOLLOWS:

i) COLEBROOK RESERVOIR:

FROM ACE "CONNECTICUT RIVER BASIN RESERVOIR REGULATION MASTER MANUAL" - APPENDIX J - FARMINGTON RIVER WATERSHED - JUNE 1970 AND/OR "COLEBROOK RIVER DAM & RESERVOIR" DESIGN MEMO. No. 2 - HYDROLOGY, NOV. 1963.

a,) INFLOW: PMF $= (Q_{P_1})_{CR} = 165000^{CFS}$ ½ PMF $= (Q_{P_1})^{*}_{CR} = 82500^{CFS}$

b,) OUTFLOW®: $(Q_{P_3})_{CR} = \underline{\underline{96000}}^{CFS}$ $(Q_{P_3})^{*}_{CR} = \underline{\underline{16000}}^{CFS}$

®(ASSUMED THE OUTFLOW FROM GOODWIN DAM - IMMEDIATELY D/S)

NOTE: THE COLEBROOK RESERVOIR OUTFLOWS FOR PMF AND ½ PMF (ASSUMED APPROX. EQUAL TO SPF) WERE FURNISHED FROM THE FIRST REFERENCE ABOVE BY NED-ACE, FOR THE FLOODS ROUTED ON THE CONDITION OF RESERVOIR EMPTY TO PERMANENT POOL AT THE BEGINNING OF THE FLOOD.

ii) MAD RIVER RESERVOIR:

FROM ACE "MAD RIVER DAM & RESERVOIR" - DESIGN MEMO No. 1 HYDROLOGY AND HYDRAULIC ANALYSIS - OCT. 1960

a,) INFLOW: PMF $= (Q_{P_1})_{MR} = 30000^{CFS}$ ½ PMF $= (Q_{P_1})^{*}_{MR} = 15000^{CFS}$ ($\cong$ SPF $= 15100^{CFS}$)

b,) OUTFLOW: $(Q_{P_3})_{MR} \approx \underline{\underline{13000}}^{CFS}$ $(Q_{P_3})^{*}_{MR} \approx \underline{\underline{430}}^{CFS}$

NOTE: THE MAD RIVER RESERVOIR OUTFLOWS HAVE BEEN ESTIMATED

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION Sheet *D-15 of 17
Computed By HM Checked By: TS Date 7/16/79
Field Book Ref. Other Refs. CE #27-595-KB Revisions

*ATTACHMENT TO COLLINS CO.
LOWER DAM 4/4 INSPECTION

*COLLINS CO. UPPER DAM

1, d - Cont'd) PEAK INFLOW

BY AN APPROXIMATE ROUTING OF THE FLOODS ON THE CONDITION OF RESERVOIR EMPTY TO PERMANENT POOL AT THE BEGINNING OF THE FLOOD. IT SHOULD BE NOTED THAT THE OUTFLOW FOR ½ PMF IS DISCHARGED TOTALLY THROUGH THE UNGATED CONDUIT OUTLET.

iii) SUCKER BROOK RESERVOIR AND HIGHLAND LAKE:

FROM ACE - HIGHLAND LAKE DAM, CT 00106 - PHASE I INSPECTION REPORT, DATED JUNE 1979 - WHICH IN TURN ASSUMED SUCKER BROOK RESERVOIR EMPTY AT THE BEGINNING OF THE TEST FLOOD.

a,) INFLOW TO SUCKER BROOK:

$PMF = (Q_{P_1})_{SB} \doteq 6500^{CFS}$ $\quad$ $½PMF = (Q_{P_1})^*_{SB} \doteq 3250^{CFS}$

b,) INFLOW TO HIGHLAND (FROM SUCKER BROOK & DIRECT D.A.):

$PMF = (Q_{P_1})_{HL} \doteq 9500^{CFS}$ $\quad$ "½PMF" $= (Q_{P_1})^*_{HL} \doteq 3600^{CFS}$

c,) OUTFLOW FROM HIGHLAND:

$(Q_{P_3})_{HL} \doteq \underline{\underline{6000}}^{CFS}$ $\quad$ $(Q_{P_3})^*_{HL} \doteq \underline{\underline{2000}}^{CFS}$

iv) SAVILLE (BARKHAMSTED) AND RICHARD'S CORNER (COMPENSATING) DAMS/RES.:

FROM ACE - SAVILLE DAM, CT 00376 AND RICHARD'S CORNER DAM, CT 00371 - PHASE I INSPECTION REPORTS, DATED SEPTEMBER, 1978, WHERE ROUTED OUTFLOWS FOR FULL PMF WERE DERIVED. APPROX. ROUTED OUTFLOWS FOR ½ PMF WERE ESTIMATED FROM DATA ON THESE REPORTS.

a,) INFLOW TO SAVILLE (BARKHAMSTED):

$PMF = (Q_{P_1})_{SR} = 78900^{CFS}$ $\quad$ $½PMF \doteq (Q_{P_1})^*_{SR} \doteq 39500^{CFS}$

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION Sheet *D-16 of 17
Computed By HM Checked By JS Date 7/17/79
Field Book Ref. Other Refs. CE#27-595-KB Revisions

*ATTACHMENT TO COLLINS CO. LOWER DAM P/A INSPECTION

*COLLINS CO. UPPER DAM

1,d,iv-Cnt'd) PEAK INFLOW - (SAVILLE AND RICHARD'S CORNER DAMS)

b,) INFLOW TO RICHARD'S CORNER (COMPENSATING), FROM SAVILLE AND DIRECT D.A.:

PMF = $(Q_{P_1})_{RC} \approx 28200^{cfs}$ "½ PMF" = $(Q_{P_1})^*_{RC} \approx 19800^{cfs}$

C,) OUTFLOW FROM RICHARD'S CORNER:

● $(Q_{P_3})_{RC} \approx \underline{\underline{26400}}^{cfs}$ $(Q_{P_3})^*_{RC} \approx \underline{\underline{17900}}^{cfs}$

● NOTE: FROM C.E. APPROX. ROUTING. - REPORT GIVES $(Q_{P_3})_4 = 24360^{cfs}$

v) NEPAUG RESERVOIR

FROM ACE-NEPAUG DAM, CT 00370 AND PHELPS BROOK DAM, CT 00378 - PHASE I INSPECTION REPORT, DATED SEPTEMBER 1978, WHERE ROUTED OUTFLOW FROM THE NEPAUG RESERVOIR FOR FULL PMF WAS DERIVED. THE APPROXIMATED ROUTED OUTFLOW FOR ½ PMF WAS ESTIMATED ALSO FROM DATA ON THE PHASE I REPORT.

a,) INFLOW: PMF = $(Q_{P_1})_{NR} = 35300^{cfs}$ ½ PMF = $(Q_{P_1})^*_{NR} \approx 17700^{cfs}$

b,) OUTFLOW: $(Q_{P_3})_{NR} = \underline{\underline{23000}}^{cfs}$ $(Q_{P_3})^*_{NR} \approx \underline{\underline{10000}}^{cfs}$

vi) CONTRIBUTION TO PEAK FROM DIRECT D.A. TO COLLINS CO. UPPER DAM:

$$PMF = (Q_{P_1})_{CC} \approx 123 \times \overset{\circledast}{600} = \underline{73800}^{cfs}$$

$$½ PMF = (Q_{P_1})^*_{CC} \approx \underline{36900}^{cfs}$$

⊛ NOTE: 600 cfs/sq mi IS UNIT PMF FOR THE TOTAL D.A. (see p. D-2)

*ATTACHMENT TO COLLINS CO. LOWER DAM H/H INSPECTION

*COLLINS CO. UPPER DAM

1,d-CONT'D) PEAK INFLOW

VII) PEAK INFLOW AT COLLINSVILLE (COLLINS CO. UPPER/LOWER DAMS)

a,) PMF $\simeq$ 359 × 600 $\cong$ 215000 CFS

IT SHOULD BE NOTED THAT IN THE CASE OF FLOODS OF THE ORDER OF PMF, THE UPSTREAM LAKES/RESERVOIRS HAVE VERY LITTLE EFFECT IN REDUCING THE PEAK INFLOW. THIS IS EVIDENCED BY THE FLOW Q=238000 CFS (>PMF) OBTAINED BY ADDING THE PEAK OUTFLOWS OF THE U/S RESERVOIRS AND THE DIRECT D.A. FLOW. THE ASSUMPTION THAT ALL THE RESERVOIR OUTFLOWS PEAK SIMULTANEOUSLY IS SO CONSERVATIVE IN THIS CASE AS TO MAKE THE ERROR INVOLVED TO BE OF THE ORDER OF MAGNITUDE OF (AND EVEN UPSET) THE EXPECTED PEAK FLOW REDUCTION.

b,) "½ PMF" $\simeq \Sigma(Q_{P_3})^* + (Q_{P_1})^*_{CC} \cong$ 83000 CFS

THIS FIGURE, ALSO CONSERVATIVE BY ASSUMING SIMULTANEOUS PEAKING OF THE PARTIAL OUTFLOWS (VALLEY STORAGE IS NEGLIGIBLE), SHOWS A SUBSTANTIAL EFFECT IN THE PEAK REDUCTION BY THE RESERVOIRS (½ PMF $\cong$ 359 × 300 $\cong$ 108000 CFS ∴ ΔQ $\cong$ 25000 CFS).

A CHECK OF THE PEAK FLOW PRODUCED BY THE DIRECT D.A. TO COLLINS CO. DAMS SHOWS $Q_{PMF} \cong$ 148000 CFS, A PEAK OF LESSER MAGNITUDE THAN THE ABOVE.

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

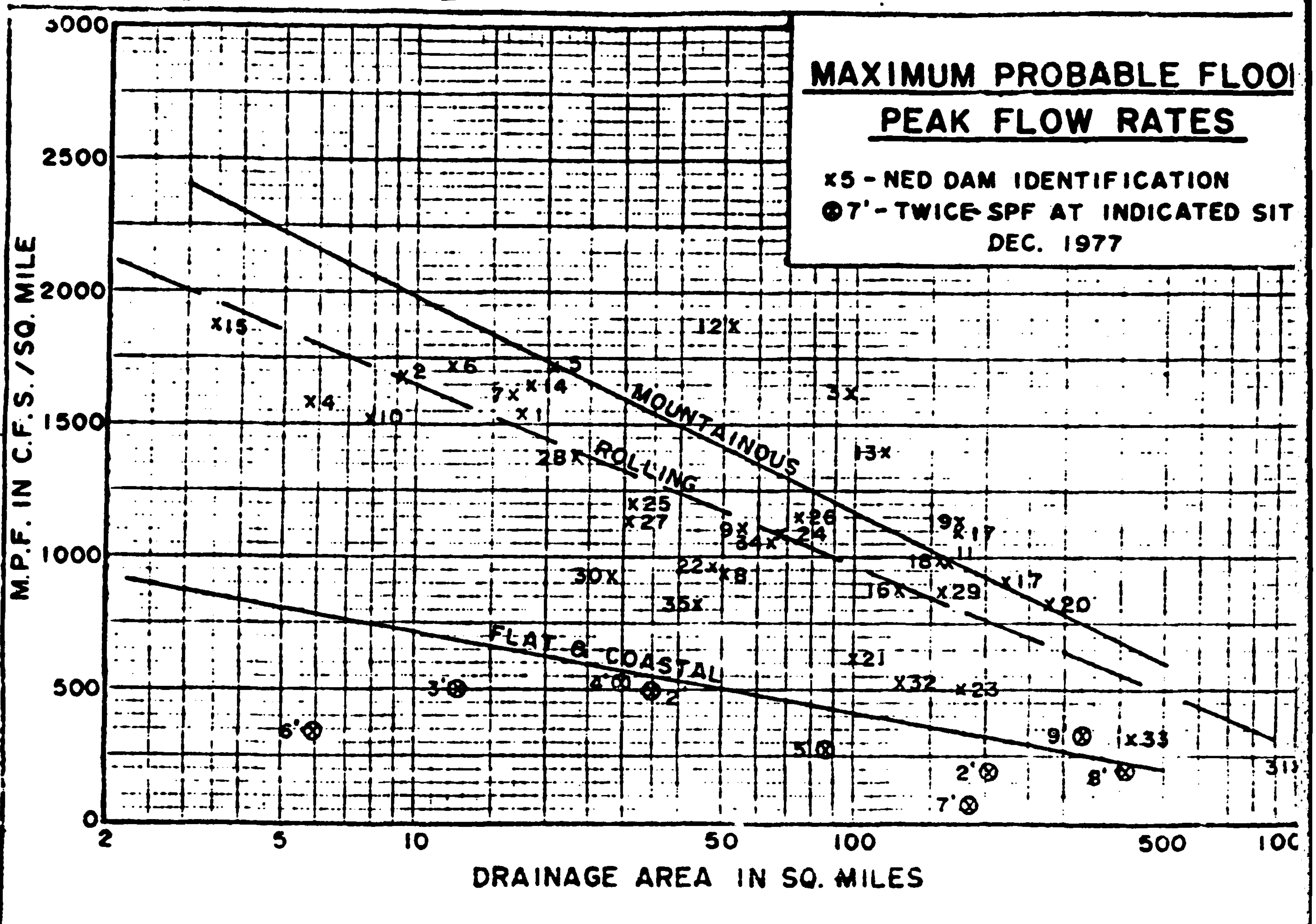
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

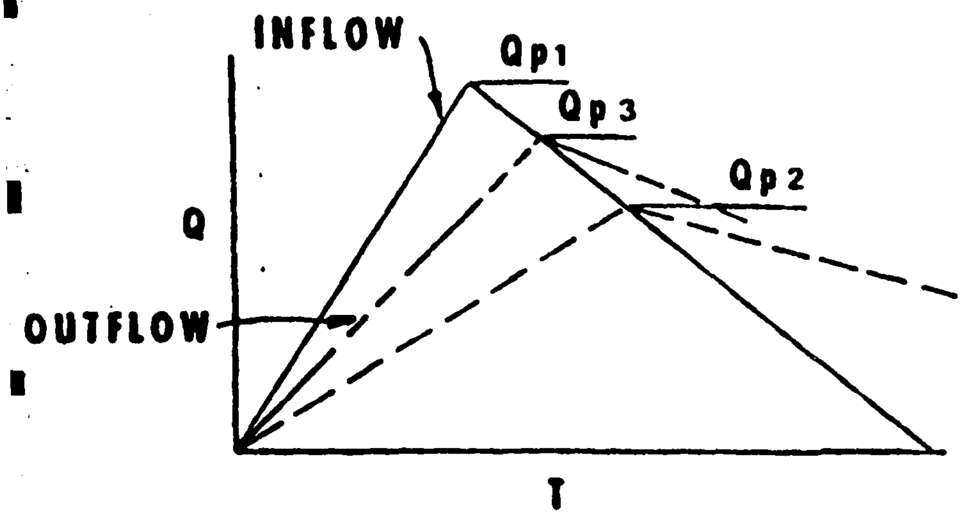
MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330



MAXIMUM PROBABLE FLOO
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SIT
DEC. 1977
MOUNTAINOUS
ROLLING
FLAT & COASTAL
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
100

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

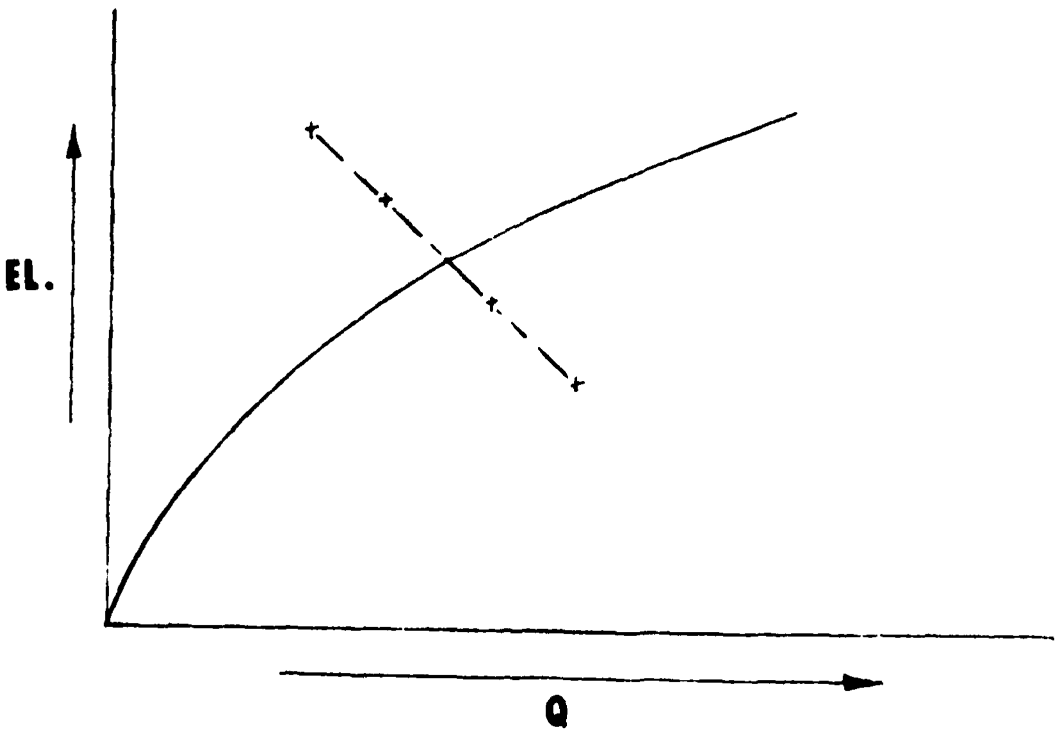
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

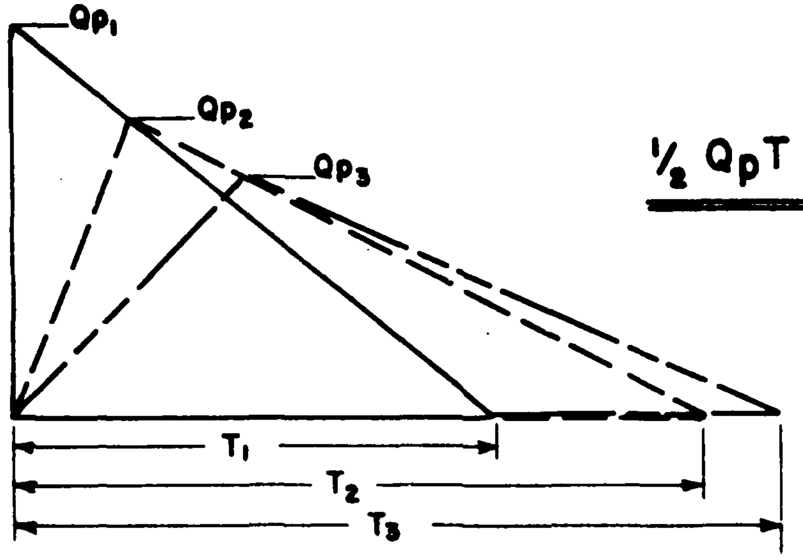
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\,S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\, Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{AVG}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS